California High-Speed Train Project

Request for Proposal for Design-Build Services

RFP No.: HSR 11-16
Structures Report
Clinton Ave to South of Santa Clara St

Prepared for

CALIFORNIA High-Speed Rail Authority

CALIFORNIA HIGH-SPEED TRAIN

Engineering Report

FINAL

Fresno to Bakersfield

U-Troughs, Bridges, and Elevated Structures

December 2011

CALIFORNIA
High-Speed Rail Authority

California High-Speed Train Project

U-Troughs, Bridges, and Elevated Structures

Prepared by:

URS/HMM/Arup Joint Venture

December 2011

Contents **Page**

1.0 Introduction

The report identifies the key features of each of the structures in Package 1A and 1B of the Fresno to Bakersfield Section within the Fresno to Palmdale Region of the California High-Speed Train Project (CHSTP). Details relating to structures in Package 1C are discussed in a separate report.

This report covers only the HST structures considered nonstandard or complex. The definition of nonstandard and complex structures is provided by Technical Memorandum (TM) 2.10.4 Seismic Design Criteria.

TM 2.10.4 clause 2.6 divides structures into a classification hierarchy as follows:

- Primary structures (structures that directly support the HST tracks)
- Secondary structures (all other structures)

Primary structures are subdivided by importance into the following:

- Important structures (structures designated by the Authority to be important)
- Ordinary structures (all other structures)

Primary structures are also classified by technical complexity as follows:

- Complex structures:
  Structures that have complex response during seismic events through
- Irregular geometry
- Unusual framing
- Long spans
- Unusual geologic conditions
- Close proximity to hazardous faults
- Regions of severe ground motion
- Standard structures:
  Structures that are not complex structures and comply with the pending CHSTP Design Guidelines for Standard Aerial Structures are the responsibility of the Engineering Management Team (EMT).
- Nonstandard structures:
  Structures that do not meet the requirements for either standard or complex structures

Table 1.0-1 lists the structures in Packages 1A and 1B of the Fresno to Bakersfield section of the HST and indicates their classification under the above system.

Table 1.0-1
Structures and Components

Package Reference	Primary Structure Name	Structural Component	Location or Start Station on Alignment S2	End Station on Alignment S2
1A	Fresno Grade Separation	Reinforced concrete (RC) U-trough Primary - Nonstandard	10885+00	10909+30
1A	Fresno Grade Separation	Braced RC U-trough Nonstandard	10909+30	10920+20
1A	Fresno Grade Separation	Covered trench at SJVR Northern Spur Crossing Primary - Nonstandard	10920+20	10940+05
1A	Fresno Grade Separation	Braced RC U-trough Primary - Nonstandard	10940+05	10933+80
1A	Fresno Grade Separation	Covered trench at Dry Creek Canal Crossing Primary - Nonstandard	10933+80	10934+20
1A	Fresno Grade Separation	Covered trench at SJVR Southern Spur Crossing Primary - Nonstandard	10934+20	10935+20
1A	Fresno Grade Separation	Braced RC U-trough Primary - Nonstandard	10935+20	10935+95
1A	Fresno Grade Separation	Jacked box beneath SR 180 Primary - Nonstandard	10935+95	10939+40
1A	Fresno Grade Separation	Braced RC U-trough Primary - Nonstandard	10939+40	10941+90
1A	Fresno Grade Separation	RC U-trough Primary - Nonstandard	10941+90	10970+00
1B	Fresno Street Overpass	Single span post-tensioned superstructure Primary - Nonstandard	10991+70	10992+50
1B	Tulare Street Overpass (Option)	2-span deck with precast box beams Primary - Standard	11001+53	11002+05
1B	Tulare Street Overpass (Option)	Single span railway bridge to carry the twin track Union Pacific Railway over the depressed Tulare Street Secondary Structure	(11001+53 approx, but not on HST route)	(11002+05 approx, but not on HST route)

1.1 Overall Design Assumptions for 30% Design

The amount of design undertaken at the 30% stage is limited to confirming that the design build (DB) contractor can continue to develop and design the concept proposed at 15% into a full detailed design suitable for construction.

In carrying out the 30% design, the designers have concentrated on the key aspects of the design stated in the TM for 30% design scope. These aspects are determined in many cases by satisfying the requirements of the relevant TMs.

For the U-troughs, the requirements are concerned with the following:

- Structural adequacy
- Constructability and consideration of adjacent constraints
- Technical feasibility (in some sections)

For the bridge structures, the requirements include the following:

- Structural adequacy
- Seismic performance as specified in TM 2.10.4 Seismic Design Criteria
- Interaction between track and structure to ensure that adequate provision is made for relative and absolute displacements between track and structure
- Constructability and assumed construction method

1.1.1 Structural Adequacy

For the U-trough, the designers performed preliminary calculations on a number of cross sections to demonstrate that the assumptions about section wall thickness, shoring wall thickness, and excavation sequence were reasonable.

The designers performed similar calculations for key components of the U-trough — specifically the jacked box and Dry Creek Canal culvert. The Dry Creek culvert itself is not an HST structure, but preliminary design was necessary to demonstrate that there was clearance for the U-trough to pass under the creek and for the San Joaquin Valley Railroad (SJVR) tracks to pass over the structure without compromising its hydraulic performance. These calculations are attached at Appendices B (Fresno Grade Separation), C (Fresno Street Bridge).

1.1.2 Seismic Performance

TM 2.10.4 Seismic Design Criteria gives the requirements for assessment of the seismic performance of structures. In terms of acceptability of the design, the requirements relating to seismic performance are Operability Performance Level (OPL) under the action of the Operating Basis Earthquake (OBE) and No-Collapse Level (NCL) of performance under the action of the Maximum Considered Earthquake (MCE):

- NCL at MCE:
- No collapse
- Significant yielding of reinforcing steel
- Extensive cracking and spalling of concrete but minimal loss of vertical load carrying capacity in columns
- Large permanent deflections
- OPL at OBE:
- Minimal impacts to HST operations
- No superstructure spalling onto tracks
- Minimal permanent deformations

TM 2.10.4 defines the two design-level earthquakes as follows:

- Maximum Considered Earthquake (MCE) - Ground motions corresponding to greater of:
- (1) a probabilistic spectrum based upon a 10% probability of exceedance in 100 years (i.e., a return period of 950 years with 5% damping) and
- (2) a deterministic spectrum based upon the largest median response resulting from the maximum rupture (corresponding to Mw) of any fault in the vicinity of the structure
- Operating Basis Earthquake (OBE) - Ground motions corresponding to a probabilistic spectrum based upon an 86% probability of exceedance in 100 years (i.e., a return period of 50 years with 5% damping)

Response spectra for design have been the subject of separate studies (see also Appendix A Geotechnical Design Memorandum). The engineering management team (EMT) has provided spectra from these studies for use in the 30% design. These spectra are reproduced in Figure 1.1-1.

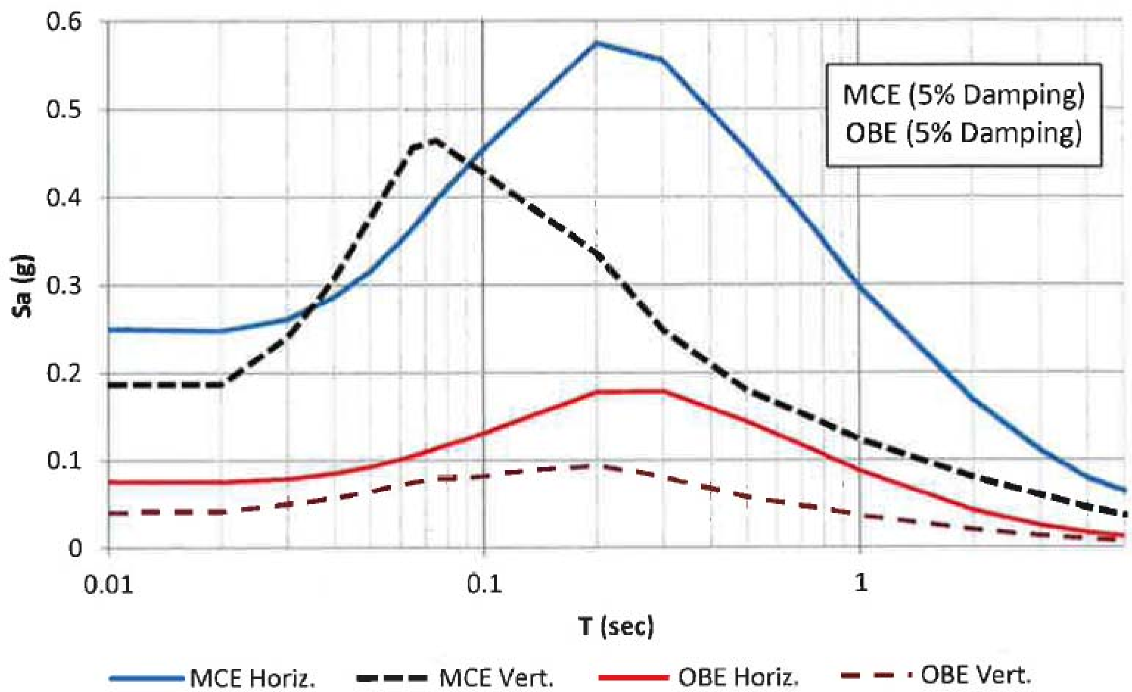


Figure 1.1-1
Design Response Spectra (Zone 4)

The seismicity in the Fresno area is categorized as Zone 4, which is the lowest category encountered on the Fresno–Bakersfield Section of the HST.

The peak ground acceleration (PGA) has been taken as the acceleration that corresponds to a period of 0.01 seconds — that is 0.0761g at OBE (red curve) and 0.2498g at MCE (blue curve). As these accelerations are less than 0.35g, in accordance with TM 2.9.10 clause 6.10.13, additional earthquake pressures can be disregarded for the design of buried structures such as the U-trough and the jacked box, provided that design for at-rest pressures is undertaken.

1.1.3 Dynamic Performance

Fundamental frequency checks have been carried out for the HST bridge structures in compliance with the requirements of TM 2.10.4 Seismic Criteria. Details of this analysis are reported in the relevant Appendix to this report.

1.1.4 Track Structure Interaction

The two HST bridges within package 1B (Fresno Street and Tulare Street) are relatively short (105 feet (32.004m) and approximately 65 feet (19.812m), respectively). It is considered that no critical interaction effect will be achieved within the length of Tulare Street Bridge and this is a Standard Structure in any case. In the Fresno Street Bridge, the design team has assessed the effect of traction and braking forces to estimate the likely movements between rails and structure. The results are reported in appendix C.

1.2 Information Required for Further Development of the Design

1.2.1 Assumptions Made for 30% Stage Design

The recommendations of the Geotechnical Design Memorandum (Appendix A) have been followed, including the following:

- Soil parameters (γ_b,$\varnothing$, c_u)
- Assumed groundwater levels
- The requirements of TM 2.3.2 (see also Section 2.2.1)

For the jacked box assessment and preliminary design, assumptions have been made concerning the following:

- Soil parameters for the State Route (SR) 180 embankment
- The percentage of ground loss (overbreak) that can be expected during excavation
- Groundwater levels

For the U-trough shoring wall, assumptions have been made concerning the following:

- The type of shoring that will be used
- Soil parameters
- Temporary construction surcharges
- Temporary brace positions, spacing, and stiffness
- The Kinder-Morgan hydrocarbon line has not been considered in the analysis undertaken; if not diverted, the permissible movements that it can tolerate (unknown at present) may influence the design and type of shoring that can be used in the vicinity

More detail concerning these assumptions is provided in the individual sections for each structure.

The DB contractor or jacking specialist should verify these assumptions based on the results of the ground investigation and any other investigation they may undertake.

1.2.2 Further Information Required to Develop the Design

It is expected that the DB contractor will wish to have more detailed information relating to the following:

- Borehole details along the length of the U-trough
- Results of soils testing (currently planned)
- Results of long-term monitoring of groundwater levels
- More detailed assessment of surcharge loading
- Detailed knowledge of access routes and timing of access to site
- Details of the location of overhead contact system (OCS) posts and wall mountings
- Detailed discussions with Fresno Irrigation District relating to timing and construction sequencing of the 96-inch storm drain diversion
- Approved schedule of road closures and durations for cross streets
- Detailed discussions with Caltrans about acceptable settlements of the SR 180 bridge in response to more detailed proposals regarding box jacking process and methodology
- Greater detail about utility crossings in order to plan the protection measures required
- Definite information from Union Pacific Railroad (UPRR) about acceptability of the Tulare Street undercrossing bridge and the methodology for installation of the new deck

The design has not allowed for temporary or permanent surcharges applied to land outside the right-of-way other than the known UPRR loadings described above. It is considered advisable that negotiations for the right-of-way should include conditions either for the permitted use of land adjacent to the U-trough that limit the loading that can be applied or that additional land is purchased so that its use can be controlled.

2.0 Fresno Grade Separation

The Fresno Grade Separation is a reinforced concrete (RC) U-trough structure that varies in depth from 0 feet (0m) to approximately 50 feet (15.24m). The trench will be constructed at a part of the route where the right-of-way width is constrained by adjacent properties; this restricts the methods by which the structure can be built, effectively excluding open cut excavation.

The grade separation structure is composed of a number of subtypes:

- Reinforced concrete U-trough
- Reinforced concrete U-trough with permanent high-level bracing
- U-trough structure with RC slab cover
- A section of RC-covered trench that is to be constructed off-line and jacked into position through an embankment
- Utility crossing structures

2.1 Structure Importance Classification

TM 2.3.2 paragraph 2.2.1 defines all structures supporting the high-speed tracks to be primary structures because they will be required to be reinstated to allow resumption of train service after an earthquake.

This classification implies the following:

- Design life is 100 years
- Seismic design must comply with TM 2.10.4; however, the seismic design criteria for the Fresno Area indicates a PGA of less than 0.35g. In accordance with TM 2.9.10 clause 6.10.13, this means that additional earthquake pressures can be disregarded for the design of this structure
- When applying the AASHTO LRFD code, values for the importance, ductility, and redundancy factors, η_I, η_D, and η_R have been chosen as follows:

- Importance factor η_I = 1.05
- Ductility factor η_D = 1.05 for strength calculations
- Redundancy factor η_R = 1.05 for nonredundant elements, 1.0 otherwise

2.2 Key Design Features and Site Constraints

The grade separation is a simple RC U-trough (where the depth is such that additional permanent bracing is not required). These sections will be designed as rigid walls in accordance with TM 2.3.2 clause 6.4.3 which means that an "at-rest" earth pressure coefficient will be used instead of an "active" pressure coefficient. Appropriate load factors from the AASHTO LRFD code will be applied to give the design forces. The typical cross section of this configuration is shown in Figure 2.2-1.

The typical section indicates a 10-foot-high wall to the left of the section (east of the route). This collision protection wall provides protection for the HST route from intrusion by derailed trains from the adjacent UPRR. This wall has been added to the structure of the U-trough because the constrained width of the right-of-way in Roeding Park and other areas precludes providing this protection on an independent foundation. This wall is not required where the separation between the UPRR right-of-way and the HST right-of-way exceeds 102 feet (31.09m).

To the right (west) side, the wall has been raised 3 feet (0.915m) above ground level to provide a nominal delineation of the edge of the trench. Additional fencing is required for fall prevention in most areas; this is not shown on the section. At the right-side boundary, access restriction fencing is provided on independent foundations to delineate the right-of-way.

Either concrete channels or swales provide drainage of the ground adjacent to the U-trough.

The section also indicates OCS equipment, which is attached either to the top of the walls or to the side faces of the wall. As the U-trough deepens, it becomes more convenient to mount the OCS on the side faces of the walls. In areas where the height of the conductor or feeder cables is within 10 feet (3.048m) of the ground, there is potential for a touching hazard. In these areas, it is considered prudent to raise the height of the wall to 10 feet (3.048m) above ground level so that the OCS can be mounted on the wall face instead of the top.

The OCS equipment is not part of the civil engineering contract; however, knowledge of its location is required in order to finalize the design of the wall in these areas.

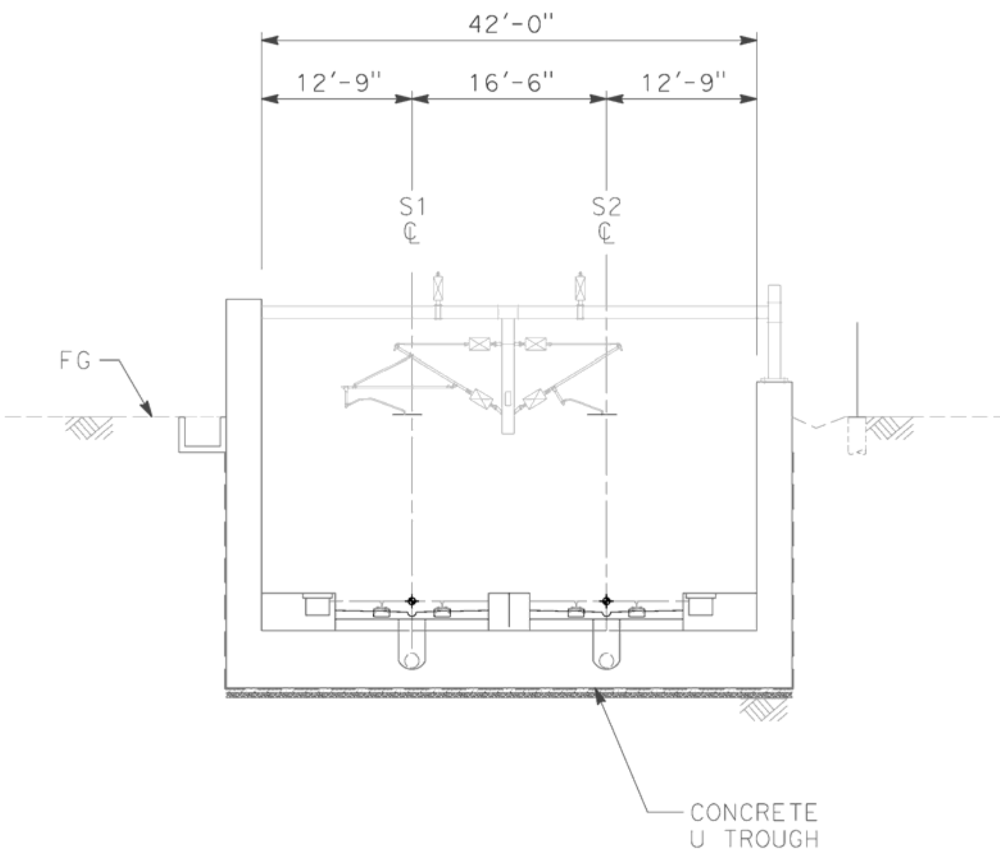


Figure 2.2-1
Typical Section of Unbraced U-Trough

At present, the ground investigation for this section of the route has not yielded any test results on which to base a design. Soil parameters used in the design have instead been based on historical geotechnical data along the HST Fresno subsection from State Routes 41, 43, and 99 as supplemented by City of Fresno residential development project records.

Where the depth of the trench exceeds approximately 30 feet (9.144m) from ground level to the top of rail, an unbraced section becomes difficult to achieve without excessively heavy reinforcement. Permanent bracing then becomes a more effective solution.

The minimum clearance requirements for the OCS system allow braces to be placed no lower than 27 feet (8.23m) above top of rail, which places the braces close to ground level at the start of the braced sections. As the trough continues to deepen, the braces maintain their clearance to the OCS. This also reduces the bending moments at the root of the wall.

At Dry Creek Canal and some utility crossings a reduced clearance of 24 feet (7.315m) has had to be provided. This is acceptable subject to approval provided that the OCS catenary supports can be arranged so that the catenary will fit under the obstacle..

Due to the additional stiffness provided by the brace, these sections must also be designed as rigid walls in accordance with TM 2.3.2 clause 6.4.3, using the “at-rest earth” pressure coefficient with appropriate load factors from the AASHTO LRFD code.

The typical section of this arrangement is shown in Figure 2.2-2.

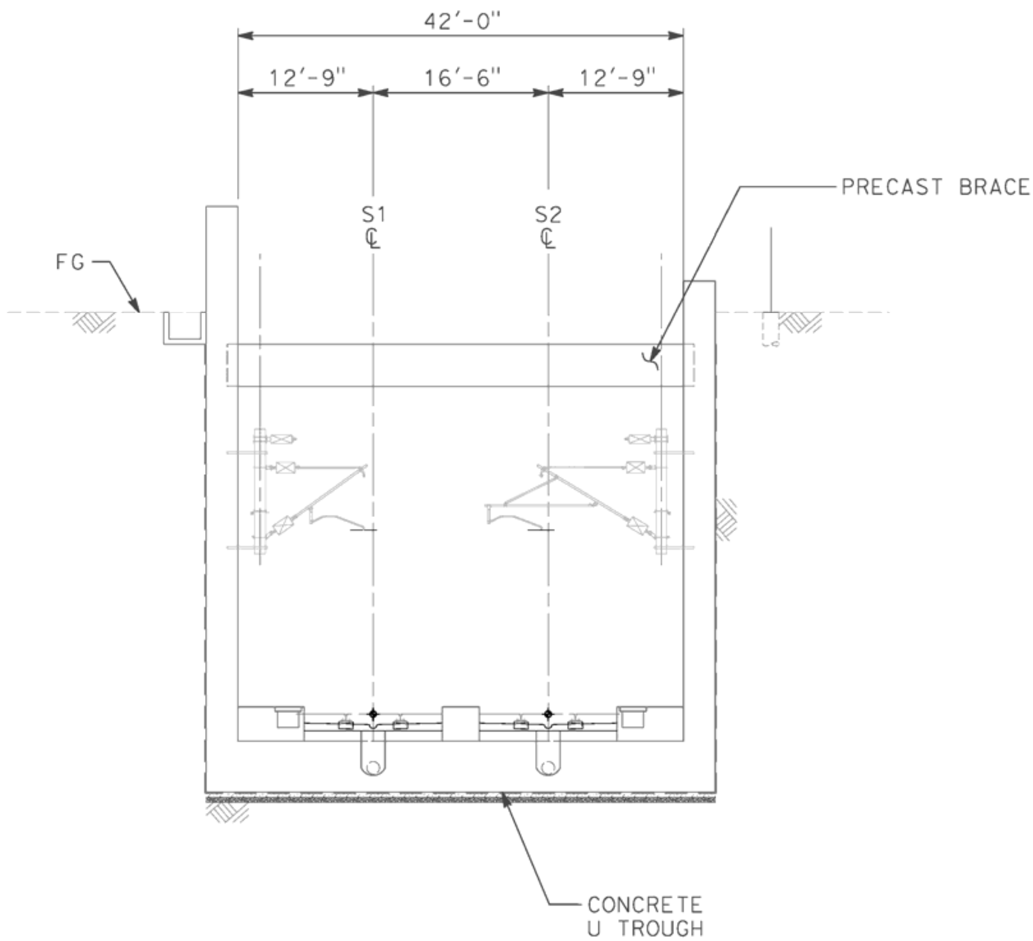


Figure 2.2-2
Typical Section of Braced U-Trough

2.2.1 Design Assumptions

2.2.1.1 Locked in Force from Shoring

In accordance with TM 2.3.4, the U-trough walls have been designed as rigid walls subject to at-rest earth pressures. In addition, where the walls will be restrained by permanent bracing, to allow for restraint forces that will be "locked-in" from the temporary bracing, the earth pressure calculated at the base of the wall has been assumed to act for the full height of the wall. This is similar to the pressures found in the design of temporary bracing to the excavation. The forces resulting from this assumption add approximately 25% to the forces that otherwise would be calculated.

2.2.1.2 Groundwater Level

Groundwater levels have been assumed to be generally below the level of the excavation except in areas where there is a ready water supply. These are assumed to be at detention basin RR2 and at Dry Creek. In these places, the water level is assumed to be 10 feet (3.048m) below ground level as recommended by the Geotechnical Design Memorandum (Appendix A). Adjacent to these areas, it is assumed that water level gradually reduces.

2.2.1.3 Surcharge Pressure

For the majority of the length of the U-trough, the right-of-way has a width of only 60 feet (18.29m). To the east side, the right-of-way of UPRR abuts the HST right-of-way, and for approximately 1000 feet (304.8m) , Roeding Park abuts to the west. Consequently, in these areas it is not likely that the construction surcharges specified in TM 2.3.2 clause 6.4.4 will be possible. Nor is it likely that future developments will add to the surcharge. In areas where the route passes between G Street and H Street, surcharges are possible because the right-of-way width is greater and because a number of properties taken by the route may include saleable parcels of land.

The UPRR tracks are generally within 30 to 100 feet (9.144m to 30.48m) of the U-trough for much of its length, and the possibility of additional surcharge from derailments exists.

At its current location, the UPRR adds little to the force applied to the wall. The maximum contact pressure of the Cooper E80 loading (driving wheels) is 1,882psf at the underside of ties. This pressure was applied to the wall using the Boussinesq formula. The resulting moment effect at the base of the stem was back-calculated to an equivalent uniform surcharge. This procedure has demonstrated that a uniform surcharge of 420psf (3.86 feet (1.177m) of fill) would be adequate allowance for the Cooper E80 load and any short-term derailment surcharge, unless the offset to the nearest track centerline is less than 20 feet (6.096m).

Where the SJVR spur tracks cross the trough, a surcharge of 1,882psf has been applied.

Similarly, where adjacent land is available for potential development, a surcharge of 600psf as required by TM 2.3.2 clause 6.4.4 has been applied.

2.2.1.4 Collision Intrusion Barriers

Containment of derailed UPRR trains is provided by increasing the height of the U-trough wall to 10 feet (3.048m). Collision forces have been considered in the design of the upper sections of the wall where forces are concentrated. The design has allowed for two forces as specified by the UIC leaflet 777-2R. In practice, the upper force of 112.4 kips applied at a 9.84-foot height is only critical in the upper sections of the collision wall itself. The lower force of 449.6 kips at a 3.24-foot height is generally critical for the upper parts of the trough wall.

In order to reduce the risk of significant impact events affecting the body of the U-Trough wall it is recommended that the wall section immediately below the collision intrusion barrier should be designed to a higher capacity so that impact effects are localized to the area above ground level.

2.2.1.5 Methods of Counteracting Buoyancy

The concept for the trough is a development of the 15% stage design concept. The concept assumes that the temporary excavation for the trough is retained by shoring walls that are either removed or abandoned after the trough is constructed. For U-trough structures like this, rising groundwater levels are a threat because of the risk that the structural will float. This has happened in some rare cases.

A number of counterstrategies were considered in the development of the design, including the following:

- **Heels**
  The directive drawings indicate a heel detail, which means that in order to float, the buoyancy forces must overcome the weight of backfill over the heel in addition to the weight of the trough itself. This detail is designed for situations where the structure is constructed in open cut or at least with greater available space. It has not been considered suitable for this trough.
- **Thick bases**
  A second way to counteract buoyancy is to make the structure heavier. This is commonly achieved by thickening of base slabs. In the case of this structure, however, it would be necessary to have base slabs over 20 feet (6.096m) thick in some locations. This would be excessively costly, both in extra concrete and in extra excavation.
- **Attachments to walls**
  When a U-trough structure has a permanent shoring wall, it is common for the U-trough structure to be connected to the shoring walls using dowels or reinforcing bars drilled and post-fixed to shoring wall. The shoring wall then resists the uplift forces from buoyancy through skin friction with the ground. In the case of this trough, this option was discounted on the basis that the directive drawings require the trough to be "fully-tanked," i.e., to have continuous waterproofing membrane around its external surface. Dowel bars or reinforcement would have punctured this membrane, compromising the seal
- **Permanent walls**
  A development of the previous option is to combine the functions of the shoring walls with that of the permanent wall. This would limit the type of wall to either secant or diaphragm walling because of the need to maintain watertightness. The base slab of the trough would be constructed as the proposed U-trough but would be doweled to the diaphragm or secant pile wall at the edges. This option has not been pursued for the same reasons as above. However, a DB contractor may wish to develop this option further.
- **Micropiles**
  This option considers the construction of Micropiles of approximately 1-foot diameter at intervals along the length of the trough. Calculations suggest that one pile 35 feet (10.668m) long under each track at intervals of 5 feet (1.524m) would be sufficient to counteract the expected buoyancy forces. This method uses approximately 1/70th of the volume of concrete that would be required by thickening the base slabs.
- **Change watertightness requirement**
  There is a clear requirement that the trough should be watertight. This is expressed in the directive drawings that require waterproof membrane. However, if this requirement were to be relaxed to permit some water inflow, it could have the following effects on the design:

Benefits

- Open up the range of wall types to include contiguous (tangent piles)
- Remove all risk of buoyancy
- Reduce the need to design for water pressures
- Remove the need for waterproofing membrane

Drawbacks

- Need to increase the size of drainage pipes and sump storage capacity and pumping
- Increased pump running cost

Risks

- At some future date if groundwater rises to a level that the inflow cannot be carried by the drainage and sumps, it may be necessary to install a cut-off grout curtain to reduce inflow or to install a pumped groundwater abstraction system, if permitted
- Retrofitting the above works would be expensive and disruptive to operations

Of the options considered, Micropiles are thought to be the most economical and effective option for restraining the U-trough.

Currently, information about actual groundwater levels is not available. Micropiles have been detailed on the drawings where water levels are expected to be highest. When groundwater information is available, this provision may be deleted or extended.

2.2.2 Key Constraints

Some constraints apply to the trench as a whole, while others are design and construction constraints that may apply to only one component of the structure. The key constraints on the trench are:

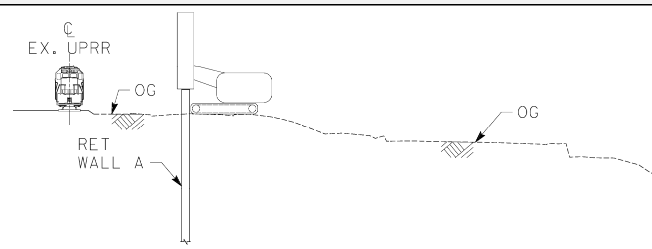
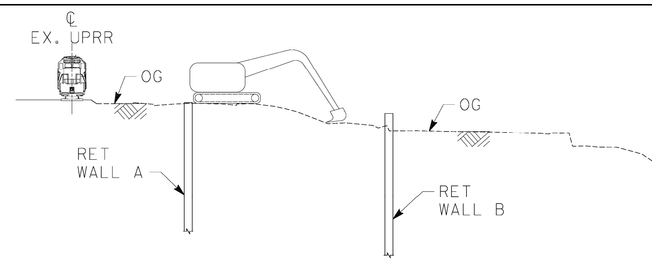
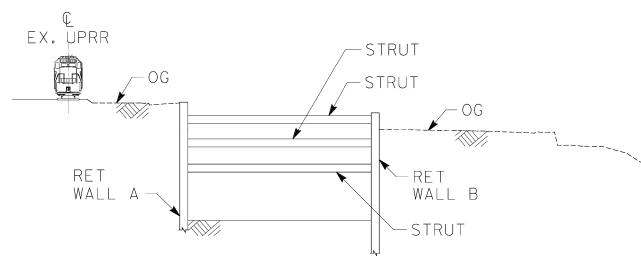
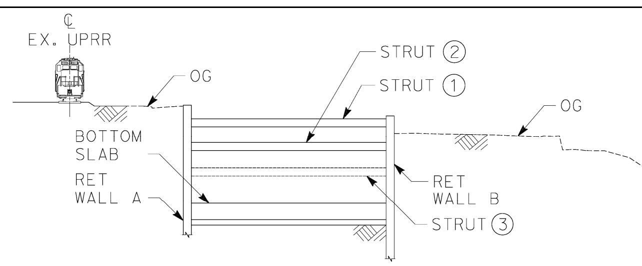
- The width of the right-of-way is generally less than 100 feet (30.48m) Adjacent to Roeding Park, it is approximately 60 feet (18.288m), and in the area of SR 180, 80 feet (24.384m). As the required minimum width for the track alignment and equipment is 42 feet (12.802m), this at worst leaves only 18 feet (5.486m) for the following:
  - All permanent retaining walls
  - Temporary shoring required for construction
  - Boundary controls required to delineate the right-of-way boundary (boundary fence, intrusion protection, intrusion detection, etc.)
  - Drainage (swales and channels)
  - The 96-inch storm drain diversion
  - Drainage sump access
  - Emergency egress stairs

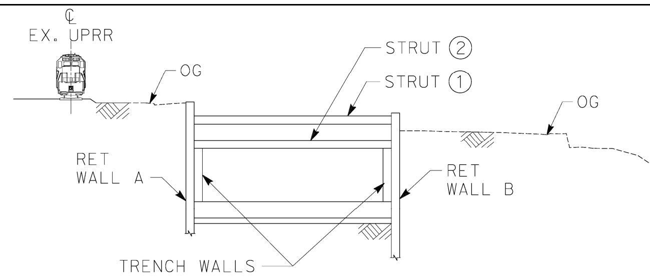
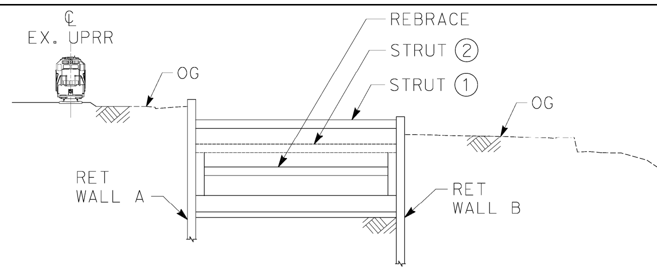
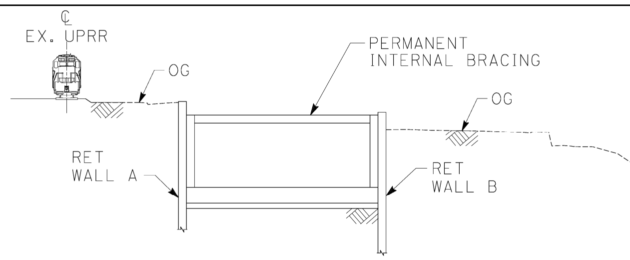
This width limitation is particularly critical in the Roeding Park area. Consequently, the method and sequence of construction of all parts of the trench should be developed in a

carefully planned sequence to avoid the risk that parts of the site may become inaccessible for the completion of subsequent operations.

The assumed construction stages are shown in the Table 2.2-1.

Table 2.2-1
Assumed Construction Stages

Construction Stage	Stage Diagram
Stage 0 Install shoring walls from within HST right-of-way.	
Stage 1 Excavate to below first brace level.	
Stages 2, 3, etc. Excavate under previous stage bracing (using low-height excavators if required). Install bracing as needed.	
At required depth Place mudmat and waterproofing membrane. Fix base reinforcement. Cast U-trough base slab.	

Stage 4 Remove lower brace (3). Fix waterproofing membrane. Fix wall reinforcement. Cast wall to part height.	
Stage 5 If required, reinstall lower brace. Repeat Stage 4 until full height achieved.	
Stage 6 Install permanent brace in deeper sections of U-trough. Cast collision wall if required.	

- The 30% design has developed this concept, and in order to confirm the feasibility of the proposed structure, calculations of the shoring wall requirements were carried out following the proposed construction sequence through to the permanent case. This work has confirmed that in principle a 3-foot wall thickness is feasible for the U-Trough, although in some locations it may need to be very heavily reinforced. In these areas, the contractor may wish propose a shoring system that can be considered as part of the permanent structure, which would allow a greater thickness of wall to be used for permanent design. However, this would require a design variance to be approved as the Design Manual prohibits this currently.
- The proximity of the UPRR tracks means that most of the length of the trench on the side adjacent to UPRR right-of-way requires the protection of a collision/intrusion barrier. The guidance provided by the project management team (PMT) was that this would be 10 feet (3.048) high and 3 feet (3.048m) thick. (Note: this may be subject to change.)
  This barrier has been added to the top of the trench wall where possible because the trench provides a foundation that is sufficiently robust to carry the accidental forces and to provide a completely separate foundation in this area would be difficult because of limited space and difficult access post-construction.
- The trench will pass partially through the edge of drainage detention basin RR2 adjacent to W Belmont Avenue. The incursion of the HST route into the basin will

result in a small reduction in the capacity of the basin.
The PMT gave direction that in this area the shoring wall should be specified as a permanent structure of contiguous bored pile construction (known as Tangent Piles). This is required to act as a first line of defense, protecting the U-trough in the event of disturbance to the basin slopes and providing additional lateral restraint to the U-trough.

- Also in basin RR2 near W Belmont Avenue, the existing storm drainage system outfalls into the basin via a 96-inch-diameter pipe that crosses the proposed HST route. The pipe must be diverted because its current invert is at a level that would conflict with the U-trough. The diversion route is shown on the utilities drawings and the structures drawings where it runs alongside the U-trough for approximately 500 feet (152.4m). The timing of its removal and construction of the diversion will be a critical aspect for coordination of construction and scheduling of the works in this area.
- The SJVR departs from the UPRR at two points. Both spur tracks cross the proposed route of the HST. To allow for this, short sections of covered trench have been designed. In order to maintain operational usage of the SJVR during construction, these sections of trench should be constructed at different times. Constructing the southern crossing first may ease the accessibility for construction of the northern crossing.
- Dry Creek Canal crosses the proposed route close to the location that the southern SJVR spur also crosses Dry Creek Canal. Consequently, the existing bridge that carries the SJVR over Dry Creek Canal must be removed. In this area, the trench has been designed as a covered U-trough.
  To provide clear separation of responsibility and ownership between the HST trough structure and the canal structure, a box culvert has been designed to cross over the HST U-trough. To ensure that the structures are separate, 1 foot of earth fill should be placed over the U-trough slab and below the base of the culvert.
  The culvert concept is a simple 2-cell RC box structure. A 2-cell structure has been selected to minimize the thickness of the top slab and therefore limit the amount of necessary vertical realignment to the SJVR while maintaining the existing soffit level.
  Initial discussions with the owners of the canal (Fresno Irrigation District) have confirmed that the concept would be broadly acceptable, subject to providing the ability to block off the cells for maintenance individually and with headwall details that match the profile of the existing canal on either end of the structure.
- The HST trench crosses under SR 180 at a point where it is on embankment. The alignment of the trench also conflicts in plan with the abutment of a bridge that takes the SR 180 over the UPRR and H Street. To avoid major disruption of SR 180, it is proposed that this section of the trench should be constructed using a box jacking technique.

2.3 Limits of Standard Bridge Design and Special Bridge Design

Standard bridge designs are not appropriate to this structure and the structure does not meet the criteria for a Special Bridge.

2.4 Construction Methods Assessment

2.4.1 Main Trench

The design team considered construction of the Fresno Grade Separation in the Constructability Memo at the 15% design stage. The 30% design has undertaken outline calculations based on assumed construction sequence to demonstrate the adequacy of the shoring system.

2.4.1.1 General Trough Excavation

The basic construction sequence described in section 2.2 and shown in Table 2.2-1 is extended slightly for the covered sections as follows:

- Construct temporary shoring walls
- Excavate to formation level, inserting temporary props as required
- Construct U-trough base slab
- Incrementally construct the side walls, removing temporary props as encountered and constructing permanent props if required by the design; where the section is covered, construct the roof lab using falsework supported from the base slab
- Backfill over the covered sections

The above sequence could be applied over the full length of the trench where the U-trough is used, or it could be implemented in discontinuous sections if routes are available for removal of excavated materials. Because access to the excavation is difficult in the middle sections, it is likely that the contractor will choose to excavate the full length of the trench prior to constructing the U-trough, except at high-risk locations.

The high-risk locations are likely to be Belmont Basin, Dry Creek Canal, and SR 180. In these locations, the design team believes that a contractor would choose to do local excavations early in the construction period to overcome accessibility problems for the remainder of the trough construction.

2.4.1.2 Roeding Park Area

At the section of U-Trough adjacent to Roeding Park, the diversion of the 96” storm drain runs parallel to the trench for approximately 500 feet (152.4m). Over this length the invert level of the storm drain rises relative to the U-Trough such that the pipe will lie alongside the U-Trough. As the width available for construction of the U-Trough and the Storm Drain is restricted, it may be necessary to vary the expected construction sequence either to install the drain and U-Trough within a shoring wall on the boundary of Roeding Park or to construct the storm drain in advance of the U-Trough and then install the shoring wall alongside the storm drain.

2.4.1.3 E Belmont Avenue

The existing East Belmont Avenue will be closed temporarily in order to construct a new overcrossing bridge structure. Once the overcrossing is constructed, the road would be re-opened and the U-Trough structure constructed beneath it. However, when the bridge beams for the overcrossing are installed there may be insufficient vertical clearance for normal piling equipment, in which case low height equipment may be necessary. However, if the shoring wall can be constructed before the beams are installed this constraint can be avoided.

2.4.2 Jacked Box Concept and Constructability

Box and structure jacking has been used in many parts of the world over the last 50 years. It has become a well-established and successful technique in that time. In practice, there are many different forms and methods of jacking that can be used. Many of the techniques used are covered by patents, and as a result, it is likely that the successful DB contractor will employ a specialist subcontractor for this work who uses only one technique.

2.4.2.1 Description of the Structure

Where the jacked box is to be constructed, the proposed right-of-way has been increased to 80 feet (23.384m) because the excavation shoring walls would be constructed farther apart than in the other parts of the excavation to allow sufficient working space for construction of the box. As the excavation must be unbraced to allow space for constructing the box, it is likely that the shoring walls will also be more substantial in this area than in other parts of the U-trough. It is expected that the contractor will wish to extend the shoring to permit the construction of

overhead braces that clear the top of the box (due to the topography of the area, the top of the box projects above ground level in the launch pit).

The box would be constructed on a base slab that is used to provide the reaction force against the jacks. This "jacking base slab" is also likely to be dowelled to the shoring wall to further distribute the jacking forces. Depending on the method used by the contractor, it is also possible that the jacking base slab will be extended part way up the sides of the jacked box as a way of providing lateral guidance to the box to ensure it stays properly aligned in the early (critical) stages of the jacking operation.

The box has been assumed to be a monolithic reinforced concrete section, though it is also possible that the contractor may choose to divide the box into segments with "interstage" jacking between segments.

The preliminary design has assumed the following key dimensions for the box:

- Length (excluding shield): 240 feet (73.152m)
- Thickness of the walls, roof, and base: 5 feet (1.524m)
- External width of the box: 53 feet (16.155m)
- External height of the box: 42 feet (12.802m)

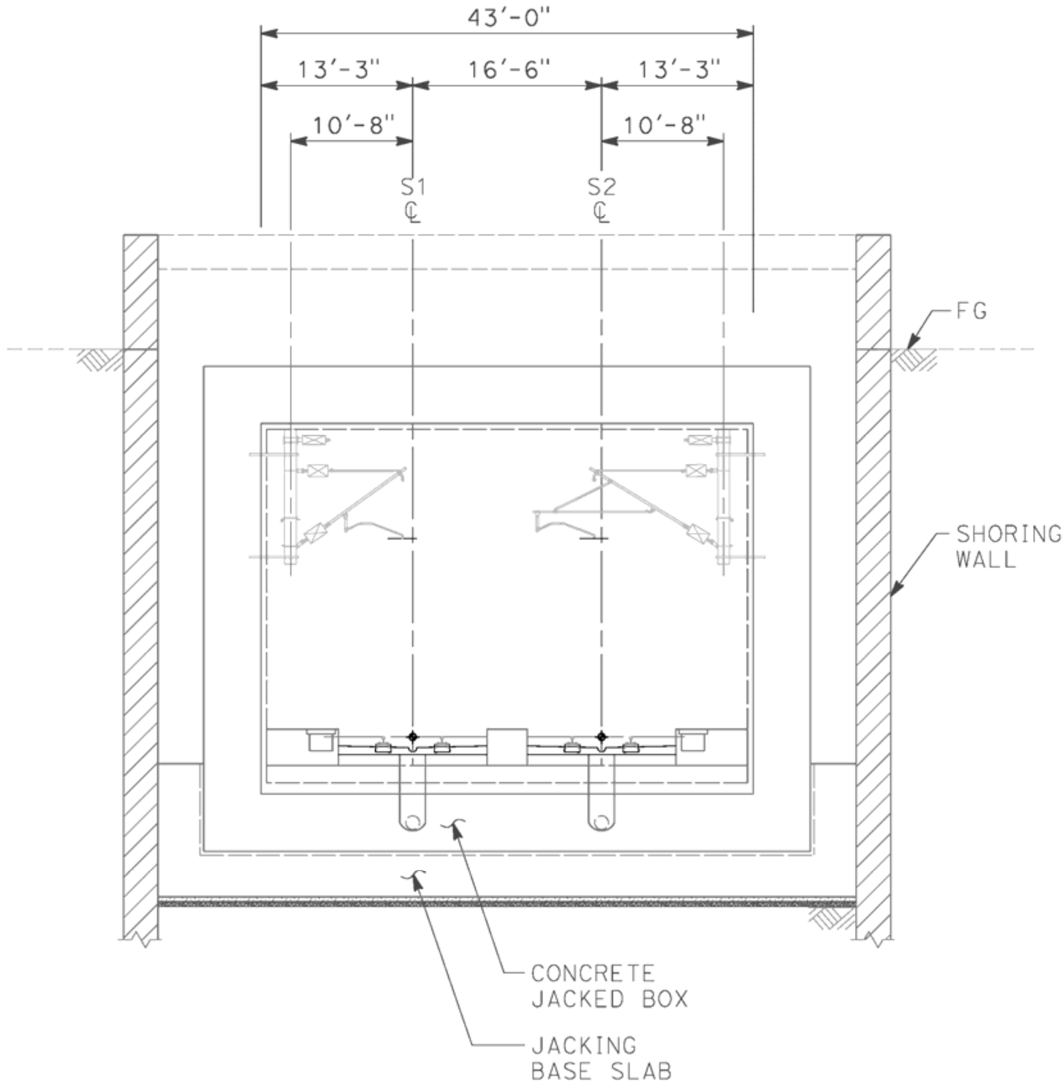


Figure 2.4-1
Cross Section of Launch Pit with Box in Position

A view showing a similar-sized box structure under construction in this situation is shown in Figure 2.4-2.

Figure 2.4-2
Example of a Partially Constructed Box in Trench Prior to Jacking

At the leading edge of the box, a purpose-designed tunnel shield would be cast on to the normal wall of the box. This will incorporate a steel cutting edge that may also be adjustable as a method of steering the box during jacking. At the rear of the box, additional fixtures may be added to accommodate the thrust jacks. A typical cutting edge is shown in Figure 2.4-3.

Figure 2.4-3
View of Shield and Cutting Edge

Note: In this case, there is no roof slab as the excavation will be open-topped.

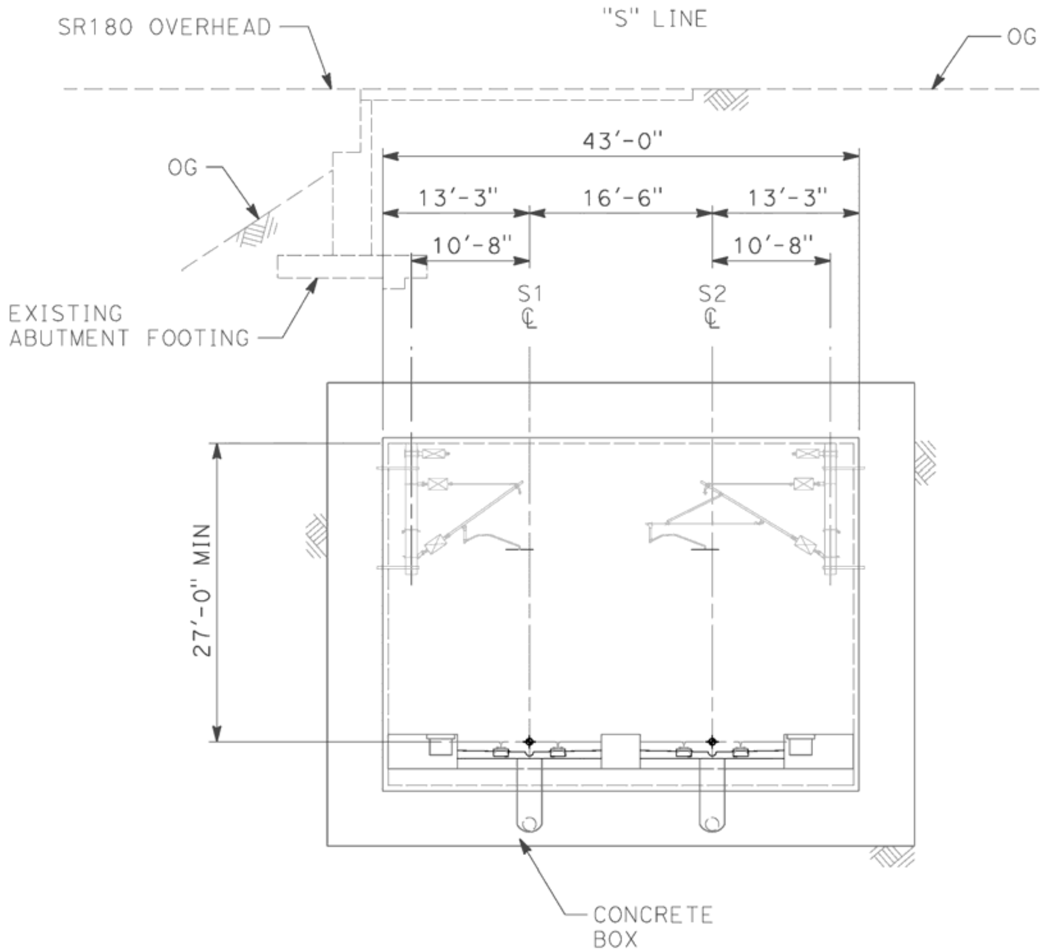


Figure 2.4-4
Cross Section and Key Dimensions of Jacked Box with Indicative Relationship to SR 180 Bridge Abutment

2.4.3 Anti-Drag System for Box Jacking

An essential component of the box jacking system is the method by which drag from the structure is reduced. This is required because as the box is jacked forward, there is a tendency for the box to drag the overlying ground along with it. In large embankments, there is some resistance to the drag force from the shear resistance of the embankment itself. However, this resistance may be insufficient to restrain the effect in the case of a wide box with low cover. If unrestrained, the ground on top of the box would be dragged forward, causing major disturbance and possible disruption to the overlying infrastructure.

The anti-drag system (ADS) is designed to prevent this — its use makes it feasible to consider box jacking where the depth of cover is as low as 6 feet (1.828m). The action of an ADS is illustrated in Figure 2.4-5.

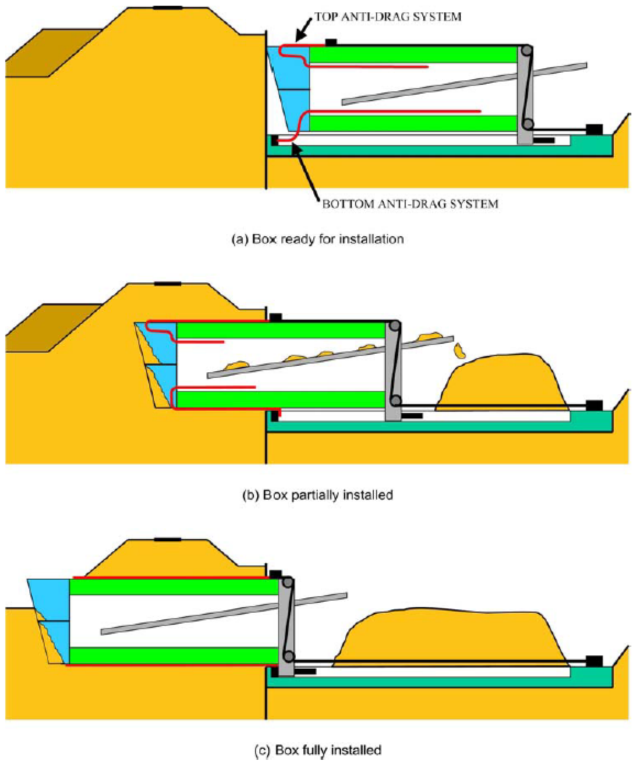


Figure 2.4-5
Illustration of Use of Anti-Drag System in Excavation

Figure 2.4-6
Anti-Drag Cables Laid Out Prior to Commencement of Jacking

One proprietary ADS comprises arrays of closely spaced greased wire ropes. The lower ADS wires are anchored to the jacking base, with their free ends passed through guide holes in the shield and stored with their free ends inside the box (the lower red line in Figure 2.4-5). As the box advances, the ropes are progressively drawn out through the guide holes in the shield and form a stationary (anchored) layer between the moving box and the ground below. The jacking forces are absorbed by the ADS and transferred back into the jacking base by the wires.

The upper ADS wires are anchored to a frame above the box with their free ends passed through guide holes in the shield and stored inside the box (the upper red line in Figure 2.4-5). As the box advances, the wires are drawn out through the guide holes to form a stationary layer that is anchored to the frame and isolates the ground above the structure from the jacking force. The wires transmit the jacking force back to the anchor frame.

In this manner the ground above and below the box is isolated from the drag forces and remains largely undisturbed.

Other systems that provide anti-drag capability follow the same basic principle but may substitute steel strips for the wires described here or use scrap conveyor belting to fulfill the same function.

The ADS wires do not isolate the sides of the box from the jacking force, so it is necessary to provide a method of reducing the frictional resistance of the sides to ensure that the force transmitted to the ground at the sides is minimized. Ground drag on the sides of the box is usually reduced by arranging the cutting edge so that a slightly larger hole is excavated than the box dimensions. Typically, the excavation is oversized by about 1 inch. However, the amount of over excavation has an effect on the amount of settlement that is seen at the surface, so overdig should be kept to the minimum necessary. Ground drag can also be reduced by lubricating the ground/structure interface with bentonite slurry. Usually both these methods are used together.

To provide lubrication, slurry injection tubes would be cast into the walls of the box during construction. These tubes would be connected to a master valve linked to the bentonite supply pipe. Figure 2.4-7 shows a set of bentonite injectors arranged in a wall that is ready for concreting.

Figure 2.4-7
Bentonite Slurry Injection Tubes

On completion of the jack, the bentonite injection tubes would be filled with cement grout to make a permanent seal.

The arrangement described above (with the exception of the upper ADS wires) is shown in Figure 2.4-8, which was taken at a recent railway project in the United Kingdom.

Figure 2.4-8
View of Jacking Area with Anti-Drag Restraints at Bottom

2.4.4 Methodology for Jacking

The design team has developed a methodology to jack the box into place. This methodology has been discussed with a specialist jacking contractor, who has commented on the methodology and confirmed that it is feasible.

This methodology is as follows:

- Prior to construction of the jacked box, construct a structural base slab. This slab is designed to guide the box during jacking and provide a reaction base against which the jacking force can be applied.
- Lay a lubricated layer of sheeting on the jacking slab. This sheeting and lubricant could be a variety of materials, but the contractor consulted preferred steel plates as sheeting because less rigid materials have a tendency to ripple and jam the jacks. The concrete box section will be constructed on this layer.
- Construct the concrete box using normal RC techniques. Because of the need to construct the box in the bottom of the trench, it may be difficult to prop the shoring walls in this area. Surveyed ground levels indicate that the roof of the box will be above ground level in this location. Consequently, the design has assumed that a more substantial shoring wall section that requires no bracing within the height of the box would be used. It is also possible that the shoring wall could be extended to a higher level so that bracing could pass over the box.
- Prior to commencing jacking, it may be necessary to undertake ground improvement to the embankment fill that the box will pass through and beneath the bridge abutment to ensure that settlements of the SR 180 abutment remain within specified limits.
  Grouting may be necessary only to ensure that the excavation face is stable and provides enough support to the upper layers of the embankment.
  Grouting may need to be more extensive to limit the amount of settlement experienced at the surface if the embankment materials are particularly sensitive to disturbance. The amount of ground improvement needed depends on the settlement tolerance specified.
  It may also be necessary to use a multicellular face shield so that the excavation face is limited to smaller pockets that can be supported individually and excavated independently.
- Once constructed, jack the box against the shoring wall that closes the end of the trench; this is expected to be broken out from within the box. Apply the jacking force through jacks reacting against the jacking slab at the rear of the box. Concrete spacer blocks may be used to adjust the jack location as work proceeds. Additional lateral support to the shoring wall will be required to ensure stability after cutting off the lower part of the shoring wall within the box.
- Continue jacking the box and excavating the face from within the box. The jacking force may be reduced by the injection of bentonite or other lubricants between the outer face of the walls and the ground as work proceeds.
- On completion of the jacking, the cutting edge and face shield will be broken out to a point where they can be incorporated into the permanent trench walls.
- Decommission the jacking pit and complete trench construction by constructing a standard trench cross section within it.
- Backfill the space between the temporary shoring wall and the finished U-trough.

2.4.5 Support for the Excavated Face

Excavation of jacked boxes of this nature requires a balance between the rate of excavation of the material that the box is passing through and the rate at which the jacking force advances the

box into the material. A secondary concern is that the excavation face will collapse in an uncontrolled way leading to over-break at the edges of the box. This may lead to the migration of material from outside the excavation zone into the excavation, which eventually results in excessive settlement at the surface. In the worst case, this might result in collapse of the overburden materials into the excavation (see Figure 2.4-9).

The above sequence is more likely to occur in loose granular materials than in stiff cohesive materials. The SR 180 embankment is assumed to be constructed of well-compacted granular materials similar in nature to the in situ ground. This reduces the risk of collapse of the face.

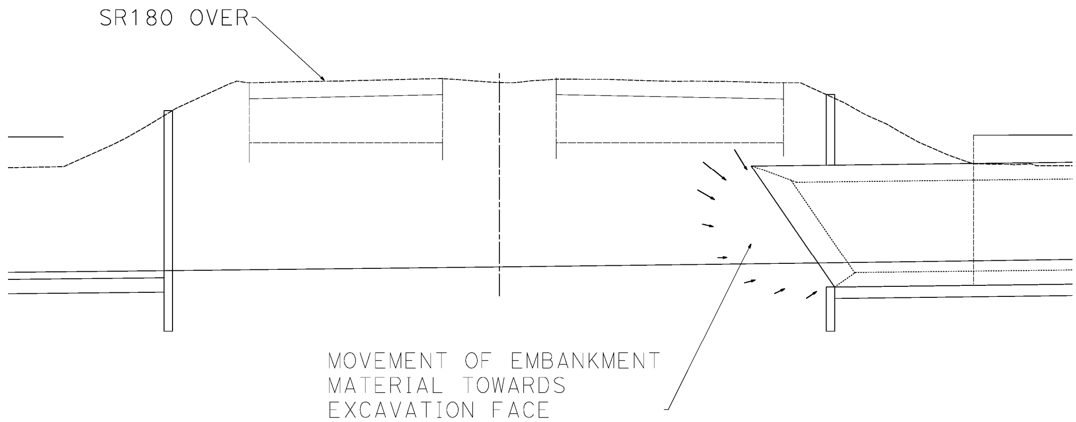


Figure 2.4-9
Excavation Process During Jacking

There are a number of ways to mitigate the risk of face collapse:

- **Pre-excavation grouting**
  The use of either chemical or cementitious grouts to increase the adhesion between the soil particles so that the excavated face behaves as a uniform, stiff, self-supporting mass during excavation. Grouting can be done either from the surface along the line of the jack or at intervals during excavation from the excavated face.
- **Compartmented excavation faces with support panels**
  The excavation size proposed for this box is approximately 53 feet (16.154m) wide by 42 feet (12.802m) high. This would present a large excavation face that may be difficult to control. In poor ground, it is common practice to subdivide the excavation face into several compartments that can be excavated by mini-excavator or by hand. This method gives the ability to control the excavation by selectively excavating certain compartments at different rates in order to steer the box and maintain directional control. Some contractors also use doors that retain the face when not being excavated. These may be hydraulically controlled and linked to the main jacks to ensure a constant pressure is exerted on the face.
- **Ground freezing**
  As an alternative to chemical or cementitious grouting, ground freezing increases the uniformity and cohesion of the excavated face by using the intergranular groundwater to bind the soil particles together for excavation. This technique is most commonly used where the excavation is below groundwater level so there is an abundant supply of water. However, because the freezing of water is an expansive process, this also means that there is a risk of heave at the surface. In

extreme cases, the frozen mass can become marginally buoyant, leading to substantially increased heave.

Of the above techniques, ground freezing is considered inappropriate, as there is unlikely to be sufficient groundwater present for it to be effective.

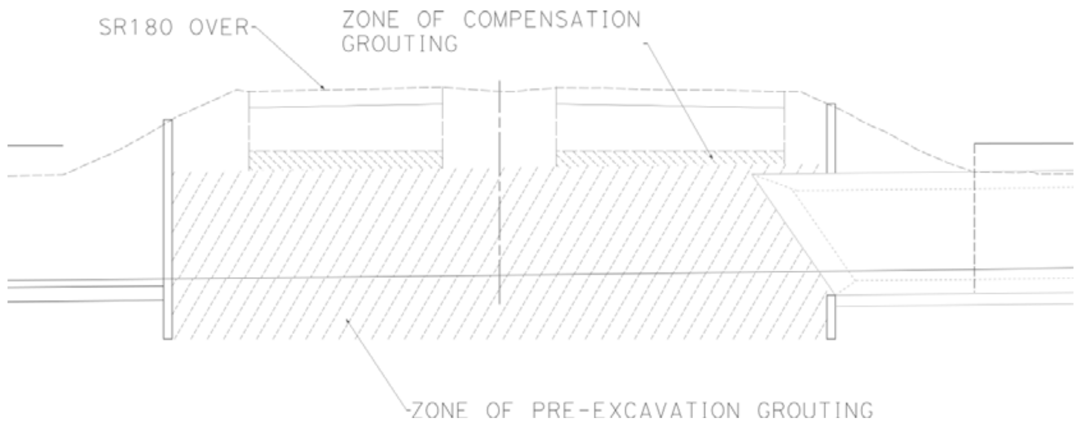


Figure 2.4-10
Zones Where Pre-Excavation Ground Treatment and Compensation Grouting May Be Used

The design team believes that the contractor will choose to use a combination of general pre-excavation grouting, compartmented excavation, and grouting from inside the box in advance of the excavated face. This combination works together quite conveniently — once excavation has started, it is possible for grouting to be done from one compartment while excavation is underway in other compartments. This also means that the excavation process can be regarded as a continuous operation. This is important as a major factor in maintaining the stability of the face comes from setting up a uniform "flow" of material through the box. If the process had to be stop/start with large time intervals between, it would be more likely to allow local collapse of weak areas, which would disrupt the uniformity of the "flow" with unpredictable results.

In cases where face collapse becomes a problem, the seemingly counterintuitive solution is often to increase the rate of excavation. This means that the calculation of required jacking force should be conservative to ensure substantial additional capacity is available if needed. In soft ground, a cellular shield configuration is normally adopted with the internal walls and decks buttressing the tunnel face. A cutting edge around the perimeter of the shield accurately cuts the hole through which the shield body and box structure pass. These cutting edges are sometimes adjustable to assist in the steering of the box. The shield provides safe access to the tunnel face for miners and machine operators, and egress for the ADSs.

The ground must have sufficient strength to arch safely across the open cells and must accept the incremental advance of the shield into it without distress. Sometimes it is necessary to improve the ground in advance of tunneling. In the ground conditions expected at the site, grouting ahead of the excavation is recommended if the water table is confirmed below the excavated profile.

Typically, 0.5 feet (0.152m) of soil would be trimmed from the face, and then the box would be jacked forward 0.5 feet (0.152m). This sequence is repeated until the tunneling operation is complete, thus maintaining the necessary support to the face.

2.4.6 Calculation of Jacking Load

The jacking load will consist of the following:

1. Reaction on shield structure
2. Friction due to the dead load of the concrete structure on the ground/concrete launch portal
3. Friction on the top and side of the concrete box against the soil

2.4.6.1 Reaction on Shield Structure

The reaction on the shield structure is assumed to be the passive pressure from the cutting edge of the shield. The thickness of the cutting edge is usually used to determine the reaction, and the resistance is calculated as the passive reaction on that area.

Based on experience on other projects, a 2-inch cutting edge around the perimeter has been assumed. Conservatively the outside perimeter is used.

$$\text{Total Area: } 2\text{in} \times (2 \times (516'' + 624'')) = 4{,}560 \text{ in}^2$$

The passive reaction is calculated at the mid-level of the box (265-feet (80.772m) above datum):

$$\sigma_P = K_P\, z\, g = 4.71 \times (320 - 265) \times 125 = 225 \text{ psi}$$

$$F = 225 \times 4560 = 1{,}026 \text{ kips}$$

A factor of safety should be applied. It is suggested to use 3.0.

$$F = 3{,}042 \text{ kips}$$

2.4.6.2 Friction Due to Dead Load

The design of the component force for the weight of the structure is based on the dead load of the structure. This should be multiplied by a suitable coefficient of friction. The coefficient of friction between concrete and steel is conservatively assumed to be 0.3.

This is an upper bound value as both within the box and within the excavated profiles, the ADS formed by a series of greased wires at the interface between the top and the bottom surface (where the largest loads from gravity are expected) will drastically reduce this contribution.

Self-weight:

$$230\text{ft} \times ((42 \times 53) - (32 \times 43))\ \text{ft}^2 \times 156\text{pcf} = 30{,}498 \text{ kips}$$

Frictional force (FF) assuming a friction coefficient of 0.3:

$$FF = 30{,}498 \text{ kips} \times 0.3 = 9{,}149 \text{ kips}$$

To include weight of shield and a factor of safety, the dead load should be multiplied by 1.2. Therefore the frictional forces would be 10,980 kips.

2.4.6.3 Friction from Soil and ADS

It is expected that the use of the ADS will impose additional forces in the jacking system. Based on references of similar projects, it is expected this load will be less than 4,500 kips.

The total load, including all the components defined above, would therefore be 18,500 kips. The maximum design single jack load is assumed to be 500 tons (1000 kips), and the number of

jacks is expected to be less than 20 units. The jacking pit and the portal structure have been verified for this load.

2.4.6.4 Ground Control

As discussed previously, the soft ground will most likely need to be pretreated to provide sufficient stand-up time during tunneling. In addition, the ground may need to be stabilized in advance to control surface settlement when tunnel jacking at such a shallow depth.

2.4.6.5 Monitoring

The jacked box tunneling operation must be carefully monitored and controlled to ensure the required performance and safety. Throughout the tunneling operation, movements at the ground surface over the area affected by the tunneling operation, jacking forces, and vertical and horizontal box alignment should be regularly monitored and compared to predicted or specified values.

2.4.6.6 Ground Settlement

The ground movements, including settlement due to the jacking of a box, are highly dependent on the method of construction, shield design, ADS's, and preparatory works. Most of the key parameters depend on the choice of temporary works, so the temporary works contractor would normally carry out the settlement assessment.

The settlement limits stated by Caltrans (see 2.4.9) relating to the abutment of the SR 180 Bridge are onerous. It is considered likely that the contractor would need to implement a compensation grouting system that will inject grout into the area below the foundation of the abutment in order to maintain or restore its original position.

In some compensation grouting schemes, the grout injection system may be linked to the movement monitoring system to automatically inject grout when the movement exceeds some defined threshold.

2.4.7 Alternative Methods of Constructing the HST Route Under SR 180

During the 15% stage of design development a number of alternative methods of constructing the HST trough in this location were studied.

These fell into two categories:

- Working under the SR 180 while in use
    - Using a jacked box
    - Propping the superstructure and using temporary bridges to carry traffic while excavating beneath to construct the U-trough
    - Extending the SR 180 bridge by adding a further span before excavating the U-trough below and through the extra span
- Closing the SR 180 in some way
    - Closing one travelway while running both directions on the other
    - Implementing full closure with diversion routes

Of these alternatives, the use of the box jacking technique was thought to be the least disruptive to Caltrans operations.

2.4.8 Summary of Feasibility Design

The 15% design assumed that the box would be constructed in the U-trough to the south side of the SR 180 embankment. This site is a building to be demolished, and consequently there is an area of land with easy road access that may be used for temporary construction. There is no equivalent to the north of the SR 180.

The 30% design has developed the requirements for jacking a box and has confirmed the following:

- A structural design for the box can be achieved that also allows for the loads from the SR 180 bridge above
- There appears to be adequate clearance between the jacked box and the SR 180 bridge foundations (based on interpretation of the as-constructed drawings)
- The jacking force required to propel the box is achievable and in keeping with that required for similar structures on other contracts
- An experienced box jacking contractor considers the proposed method achievable
- There are ground treatment techniques that would render the embankment material suitable for the controlled excavation needed for the proposed technique

2.4.9 Discussions with Caltrans about the SR 180 Bridge

The design team met with Caltrans on October 23rd 2011 to discuss the proposals for the box jacking and to determine their requirements for the following:

- Control of settlement of the SR 180 structure during the box jacking process
- Reinstatement of the bridge afterward should this be necessary

The team explained that the box would pass directly below the abutment foundation of the SR 180 bridge, and information was requested relating to permitted settlement of the structure.

Caltrans subsequently provided information that can be summarized as:

- The abutment movements must not exceed ¼ inches horizontally ½ inches vertically, whereas
- The vertical deck movement must not exceed 1 inch for continuous superstructures and 2 inches for simple spans
- All proposals relating to crossing of the SR 180 will be subject to Caltrans review and approval before work is permitted to commence

In order to comply with these movement limitations it is likely that the contractor will be required to undertake extensive grouting of the ground under the abutment. It may also be necessary to install compensation-grouting equipment linked to a settlement monitoring system to adjust the foundation of the bridge as jacking proceeds.

2.4.10 Conclusions

There are a variety of methods in use for the construction and installation of jacked structures.

It is considered highly likely that the main contractor will choose to use a specialist subcontractor that has its own preferred method of jacking. Some available methods may be unable to satisfy the movement tolerances specified and so contractors may investigate other options. The methodology described in this section is therefore considered to be “proof of concept” rather than a definitive statement of how this section of the grade separation should be accomplished.

The studies undertaken and summarized in this report have demonstrated that the methodology is feasible within the limits of the information available at this stage of scheme development.

The constraints on movement required by Caltrans make the box jacking technique more challenging but not infeasible.

Therefore, we believe that the status of the design is that it is capable of being developed into a proposal that will be acceptable to the HST Authority and to Caltrans. However, the acceptance of Caltrans cannot be regarded as certain.

2.5 Temporary Construction Loadings Considered

During the construction of the U-trough, a number of temporary construction loads will be present for short or long periods. Refer to TM 2.3.2 clause 6.4.4.

The shoring design allows for the following:

- The effect of a Cooper E80 Train set on the Union Pacific tracks adjacent to the excavation. The peak pressure of 1882psf at underside of tie level has been converted into an equivalent uniform surcharge load of 420psf applied at ground level adjacent to the wall. (See 2.2.1.)
- A surcharge pressure of 600psf has been applied to areas where construction activity may use land adjacent to the U-trough. This is not additional to the train loading above and is also not applied in areas where construction access is not permitted.
- Variable groundwater levels in the section of the trench adjacent to the Belmont Basin and in the area of the Dry Creek Canal crossing.

2.6 Temporary Construction Easements

Temporary construction easements are required for the construction of the following:

- The diverted 96-inch storm drain outfall
- Dry Creek Canal structure
- SJVR connections
- Connections to the trench drainage sump
- Emergency egress stairwells and emergency access roads

The drainage sump is located between two spur tracks and will be connected to the local drainage system via a new detention basin. The basin will be constructed adjacent to the southern SJVR spur line.

2.7 Traffic or Pedestrian Diversion and Control

The construction of the trench requires the permanent closure of W Belmont Avenue Underpass, N Thorn Avenue, and part of Golden State Boulevard. Replacement overcrossing bridges are to be provided at W Olive Street and W Belmont Avenue.

Traffic management will be necessary to accomplish these changes. The contractor will be required to coordinate and plan works in these areas so that traffic disruption is minimized to the satisfaction of the City of Fresno.

For the construction of the U-trough, there will be a need for construction entry and egress points that connect to the road system. It is expected that the majority of excavated material from the U-trough will need to be taken offsite via these egress points, so it will be necessary to

agree upon the amount, frequency, and operating hours for these entry/egress points with the City of Fresno.

2.8 Drainage Concept

The track drainage within the trench will be carried in two longitudinal pipes cast into the base slab in accordance with the directive drawings. At the low point of the U-trough (STA 10926+00), the drainage flow will be collected at a sump adjacent to the west side of the trench structure where it will be pumped to a new detention basin located within the environmental footprint adjacent to the southern SJVR spur line. Outfall from the basin will be attenuated to discharge only at the rate of a 2-year storm as discussed and agreed with the Fresno Metropolitan Flood Control District (FMFCD).

For design, it has been assumed that the depth to the natural groundwater level is around 60 feet (18.288m) below ground level, except in areas where higher or perched water levels may be expected. This assumption is based on historic borehole data from Caltrans projects in the area in the absence of more recent information.

Higher groundwater levels have been assumed to exist at the Drainage Detention Basin (RR2) adjacent to W Belmont Avenue and at the point where Dry Creek Canal crosses the HST route. In both cases groundwater has been assumed to be 10 feet (3.048m)below ground level as recommended in the Geotechnical Design Basis Memorandum (Appendix A).

A ground investigation has been commissioned, but it will not be able to provide improved data before completion of the 30% design phase.

Based on the above assumption, it is not expected that cutoff walls will be required at the ends of the trench to limit groundwater inflow. Buoyancy checks have been carried out assuming groundwater levels as above. These checks show that either additional thickness of base slab or tension piles would be required to provide the necessary factor of safety against flotation.

2.9 Emergency Egress and Escape Provision

Although not strictly an elevated or underground facility, the team has agreed that it is appropriate to apply the requirements of NFPA130 for emergency escape/egress to the U-trough. This means that escape stairwells are to be provided at maximum 2,500-foot intervals through the box. Stairwells are provided as indicated in Table 2.9-1.

Table 2.9-1
Stairwell Provisions

STA	Locale	Egress features
10906+00	Adjacent to communication site, located in the abandoned connection of Golden State Boulevard to W Belmont Avenue	Stairwell is located close to the communication site and will share a common road access track. There is space for provision of a turning area for vehicles.
10925+00	Between the north and south SJVR spur connections	Emergency services access to the location of the stairwell will need to be agreed with the owner of the facility. There is space for provision of a turning area for vehicles.
10950+00	South of Divisadero Street and adjacent to G Street	Stairwell is located in an area currently used as a vehicle parking area with a frontage onto G Street. There is space for provision of a turning area for vehicles.

Each stairwell is 10 feet (3.048m) wide by 25 feet (7.62m) long to allow for the later installation of a staircase.

The staircase is assumed to be 44in minimum width with 5-foot-wide landings at 12-foot vertical intervals and 21 treads per flight.

2.10 Inspection, Service, and Maintenance Access

The trench structure itself will be a simple massive RC structure with a limited number of movement joints at intervals. There will be no specific provision for inspection or maintenance access other than the general maintenance access to the route.

The drainage sump will require pedestrian access at the surface and access for the installation and removal of pumps. Pedestrian access will also be provided by construction of an access door from the emergency walkway within the trench. Providing this door increases the risk that it may be dislodged by the passage of a train, so it is proposed that this door and the doors associated with the emergency escape stairs should be sliding doors. These may be fitted during a later contract.

Access for pump replacements will require a permanent easement and is likely to be via the area of land between the SJVR spur tracks.

Movement joints in the walls will be required to limit the effects of temperature and ground movement. These joints are intended to be no more complex than simple cast-in waterstop details.

2.11 Utilities Affected and Disposition

A number of existing utilities cross the route of the trench or are within the proposed right-of-way. Where these can be diverted, the proposed diversion route has been identified on the utilities and structures layout drawings. It has been a principle of this work to divert utilities into new infrastructure (such as road overcrossings) or into the fill over the covered parts of the trench where possible. Where there are specific crossing points that cannot be accommodated in

this way, a utilities crossing structure is incorporated into the detail of the trench or the trench design has been modified to accommodate the utility.

Examples of where the trench design may be affected are as follows:

- **Kinder Morgan hydrocarbon line**
  This utility does not in fact enter the proposed right-of-way of the HST route. It runs along the UPRR right-of-way in an easement granted by UPRR. Its precise route varies along the right-of-way and in some places appears to be within 5 feet (1.524m) of the right-of-way. The location shown on the utilities plans is based on information provided by Kinder Morgan, but it's accuracy has not been verified by excavation.
  For the construction of the trench, care must be taken to consider the effects that the trench construction methodology will have on this utility. At this time, all that is known about the line is that its diameter is 12in.
  Concern is based on the following:
    - The pipeline has been in service for around 30 years, and its current condition is unknown to the design team.
    - Given the above, it is unknown whether the pipeline is sensitive to the magnitude of ground movement that may be expected from construction of the U-trough.
    - The pressures at which hydrocarbon lines operate are usually very high in order to minimize the number of intermediate booster stations required. Consequently, a break in the line could occur explosively and be difficult to contain.
    - The line is reported to be buried deep enough to pass under the depressed Fresno Street, which suggests it may be up to 20 feet (6.096m) deep. This depth is a further indication that the operating pressure of the line is high.
    - To the design team does not know whether the pipeline is currently leaking into the surrounding ground or has leaked in the past. The presence of hydrocarbons in the excavation would influence the choice of excavation methods that a contractor would use in the U-trough excavation.

  In order to clarify these issues Kinder Morgan were contacted and provided an initial response by e-mail on December 5 2011. The main points are:
    - Location and Depth: Exact Location & depth and only be determined by potholing. The Alignment sheets will give the general location but no depth information.
    - Type of pipe & diameter: this information is on the alignment sheets generally speaking it is steel pipe 12.75" OD. Wall thickness varies.
    - Foundation beneath the pipe not sure exactly what they are asking, usually the pipe is bedded in clean soil.
    - Contents and pressure within the pipe: liquid petroleum products (motor and jet fuels). The maximum operating pressure (MOP) is around 1440 psig however the operating and control pressures will vary along the pipeline.
    - Condition of the Pipe: the pipe meets or exceeds all regulatory requirements.
    - Date of last inspection: KM has a robust inspection program however I do not see how this information is pertinent to your design team.
    - General performance: overall good.

- Allowable movements: Lets discuss at our meeting, I need to know the context and purpose of movement.
- Design criteria: 49 CFR 195
- Support Methodology & serviceability criteria of the support: KM will determine the adequacy of any proposed supports.
- Local soil lithology: I don't believe we have the information for the hundreds of miles of pipelines that KM operates in the State.

This information confirms that the working pressure of the pipeline is likely to be high.
No clarity is provided as yet regarding tolerance to movements of the ground or proximity to the shoring walls.
It is not clear how the pipe can be protected from ground movement but it is known that the pipe is placed in the earth of the trench. So that there are no additional elements that may stiffen the pipes response to movement.

Overall the pipeline's proximity to the excavation remains a concern. Therefore, it is recommended that this pipeline be diverted to the east side of the UPRR right-of-way prior to construction of the U-trough structure.

- **96-inch storm drain outfall crossing the HST route at STA 10897+30**
  This is a diversion of the existing outfall to drainage detention basin RR2 that is located adjacent to W Belmont Avenue. The diversion of the existing facility is essential to the construction of the U-trough in this area and the diversion route that is indicated on the drawings lies in close proximity to the trench.
  After crossing under the HST route, the storm drain outfall runs parallel to the U-trough for over 500 feet (152.4m) until it reaches the detention basin. Its location, between Roeding Park and the U-trough, will be a substantial constraint on the working space available for construction of both the U-trough and the diversion. Both must therefore be considered together when developing the methodology for construction in this area.
  The vertical position of the storm drain is also a constraint in the location of the emergency escape stairwell.
- **12-inch water line at STA 10915+60**
  This is a small-diameter water supply pipe whose route cannot avoid crossing the U-trough. The diverted utility passes around the edge of the detention basin before crossing the HST via a utility crossing bridge. The utility crossing bridge will be a concrete box that will totally enclose the sleeve through which the utility pipe is installed. Future maintenance of the utility will be carried out by withdrawing the pipe from its sleeve.
  At this location there are also a number of gas lines that cross the U-Trough so that the utility crossing structure is likely to be around 10-feet (3.048m) in width.
- **30-inch sewer line at STA 10933+30**
  This is an existing gravity sewer that currently passes under Dry Creek Canal. The diversion crosses the route at a vertical clearance of 24 feet (7.315m) because a pumped solution is considered unacceptable by its owners.
- **Dry Creek Canal culvert at STA 10934+05**
  Dry Creek Canal will be culverted to pass over the trench on its current alignment and invert level. In order to maintain separation of the culvert structure from the trench structure, a minimum thickness of 1 foot of fill is to be placed between the upper surface of the cover slab and the culvert foundation.
- **12-inch Gas Line**
  This pipeline is a diversion of an existing line and passes through the fill covering the HST adjacent to the Southern SJVR spur.

- **60-inch storm drain diversion at STA 10935+85**
  This is a new storm drain that is the diversion route for a drain that currently crosses the route of the HST at Divisadero Street. It is a gravity design and crosses the U-trough in a concrete sleeve structure at a minimum vertical clearance of 24 feet (7.315m).
- **SR 180 route crossing at STA 10937+00 to 10939+50**
  It is believed that any existing utilities along the route corridor are relatively shallow. The U-trough adjacent to the SR 180 is at considerable depth. The design concept in this location is for a large concrete box to be jacked through the embankment of SR 180, passing underneath any near surface utilities and the SR 180 bridge abutment at a depth of approximately 20 to 30 feet (6.096m to 9.144m) below road surface. The utility plans indicate an abandoned oil pipeline that, from its alignment, predates the construction of the SR 180 embankment. The utility information does not indicate that the pipeline was removed during construction of the embankment, so it is assumed still present. The depth of other oil pipelines in the area suggests that this line is at approximately 10 to 15 feet (3.048m to 4.572m) below ground level, which means that it would be encountered during the excavation of the jacked box. The contractor will need to be prepared to deal with the excavation of potentially contaminated ground on the route of the pipeline.
- **20-inch water pipe at STA 10940+15**
  Similar to the other crossing, this is a service line that cannot avoid crossing the route of the HST and for which there is no reasonable alternative route. The pipe will be carried by a concrete surround and will be sleeved through the structure to permit removal and replacement.
- **Flood overflow at STA 10942+80**
  This is not strictly a utility and the purpose of this structure is discussed under hydrological issues in the next section.

2.12 Hydrological Issues

These issues are discussed in detail in the Floodplain Impact Assessment Report.

The main impact of the trench design is to ensure that the trench wall is substantially higher than the 100-year flood level in the Dry Creek Canal area. In this area, the 100-year flood level is approximately at ground level. Protection against flooding will be provided indirectly because the requirements for collision/intrusion protection require a wall 10 feet (3.048m) higher than ground level and at the west side the trench wall is 3 feet high.

During discussions with FMFCD, it was noted that the area to the south of SR 180, north of Divisadero Street, and to the east of the HST route would be cut off by the construction of the U-trough. The FMFCD has commented that in extreme flood events (50-year return period or more) this area can develop an overland flow toward the west that relieves flooding to the east. The FMFCD would like this “relief valve” to remain after construction of the U-trough. To provide for this, a closed box (similar to a utility crossing) has been added to the trench approximately at ground level. Under normal circumstances, this structure will be completely empty, but in the extreme cases described, it will allow water to flow across the HST route.

2.13 Noise Mitigation and Acoustic Treatment

No specific features have been included to mitigate the noise generated by the passage of trains. As the route is located in the trench, the trench itself will tend to direct noise generated by the passage of trains upward. This is likely to have little attenuation effect as the trench walls will be

hard and reflective to sound. The presence of collision intrusion walls between the UPRR and the trench may provide some local attenuation to generated noise due to the increase in path length.

Discussions have taken place regarding the construction of a 15-foot-high sound wall or noise protection barrier between the HST corridor and Roeding Park. The details of this wall are not finalized at this time and its construction does not currently form part of the scope of the DB contract. However, should the wall be required at a later date, the details of the U-trough wall adjacent to Roeding Park as currently developed would have no difficulty in accommodating the additional loads imposed by such a barrier.

2.14 Compliance with Systemwide Bridge Aesthetics Features

No guidance has been provided on aesthetic considerations relating to this structure form.

2.15 Details of the Geotechnical Parameters Used for Design

The geotechnical parameters are described in the Geotechnical Design Memorandum attached at Appendix A.

3.0 Fresno Street Overpass

The new Fresno HST Station is planned to be constructed at ground level. This means that to maintain highway connectivity, the cross streets near the station must go either under or over the HST tracks. Currently it is planned that Stanislaus Street and Tuolumne Street would go over the HST and Fresno Street under. Both under and over options exist for Tulare Street pending a decision by the HST Authority and the Federal Railroad Administration (FRA) on which is preferred.

At Fresno Street, an overpass (HST over) is required with a new bridge to carry the HST tracks. At the 15% stage, the bridge spanning the overpass was developed as a standard 2-span Precast Concrete box beam structure with two 40-foot spans (as Fresno Street is approximately 80 feet (24.384m) wide).

The HST Authority has subsequently agreed that the City of Fresno may allow the design and construction of this overpass as a separate contract in advance of the main works contract. This means that the overpass would have been constructed before the main works contract commences.

It is understood that the overpass structure will also be under separate ownership and maintenance responsibility than that of the HST route and consequently the HST Authority requires that the new structure is structurally independent of the overpass structure.

A further consequence of the separation of the underpass from the HST structure is that in order to avoid foundations for the HST structure in the median of the underpass and to provide adequate separation from the underpass walls, the new structure must span over the underpass in a single clear span of approximately 100 feet (30.48m).

The available vertical clearance between the HST rail level and the alignment design for Fresno Street is insufficient to allow the use of the standard viaduct box girders.

The concept was developed to satisfy the above constraints and allow the HST structure to be built with the new underpass structure in place.

3.1 Structure Importance Classification

TM 2.3.2 paragraph 2.2.1 dictates that all structures supporting the high-speed tracks are primary structures because they are required to be reinstated to allow resumption of train service after an earthquake. This classification implies the following:

- Design life is 100 years.
- Seismic design must comply with TM 2.10.4.
- When applying the AASHTO LRFD code, values for the importance, ductility, and redundancy factors, η_I, η_D, and η_R, have been chosen as
    - Importance factor η_I = 1.05
    - Ductility factor η_D =1.05 for strength calculations
    - Redundancy factor η_R = 1.05 for nonredundant elements, 1.0 otherwise

3.2 Key Design Features and Site Constraints

The structure is located in a situation where it will be slightly below existing grade and will span a new underpass. Access to the construction area will be available from within the HST right-of-way, but Fresno Street will divide the site unless traffic can be diverted to allow a temporary road closure.

To construct the in situ superstructure, the underpass must be closed for a period or the falsework required to construct the superstructure must span the underpass to allow traffic to pass underneath, possibly with reduced vertical clearance.

3.3 Limits of Standard Bridge Design and Special Bridge Design

Standard box girder designs are too deep to satisfy the vertical clearance requirements for this structure.

3.4 Construction Methods Assessment

It has been assumed that the superstructure of this bridge will be constructed in situ because the required clear span is too large for the use of standard precast box beams that might be transported by road.

It is possible that segmental precast units could be assembled to form the complete deck, but these units would require special moulds to be constructed and there would be insufficient opportunities elsewhere on the project for repeat uses to make this economical.

3.5 Temporary Construction Loadings Considered

The proposed construction sequence is as follows:

- Construct piled foundations and bearing seat on embankment behind the underpass retaining wall
- Construct the soffit falsework for the deck
- Cast the new deck
- Post-tension the deck when the concrete has reached and adequate transfer strength
- Fit the permanent bridge bearings
- Strip falsework and reopen Fresno Street
- Complete HST works

As the superstructure will be supported on falsework until stressed, no additional loads have been allowed for in the design.

3.6 Construction Easements

Temporary construction easements may be necessary to permit the closure of Fresno Street for the construction of the superstructure.

3.7 Traffic or Pedestrian Diversion and Control

Temporary closure of Fresno Street may be required if full-height falsework is necessary. If spanning falsework is used, there will be a need to restrict high vehicles, as the structure on completion will have a minimum vertical clearance of 16 feet 6 inches (5.029m).

Whichever falsework system is used, pedestrian access may be diverted to avoid safety issues from passing the public through a worksite.

3.8 Drainage Concept

The track drainage on the structure will connect to the main longitudinal drainage at the ends of the structure. Since the superstructure is only 100 feet in length (30.48m), it is not expected that drainage pipes need to be incorporated into the structure.

3.9 Inspection, Service, and Maintenance Access

The abutment structure has bankseats that include space for access. This area will be accessible for inspection from the HST right-of-way. Bearings are located on the top of short plinths so that they are clearly visible for inspectors.

The only movement joint will be at the base of the downstand ballast wall. It is intended to be a simple joint made with compressible filler board to allow movement of the structure; it is not intended to be watertight.

3.10 Utilities Affected and Disposition

It is expected that during the construction of the underpass, all affected utilities will be diverted into the underpass itself. No other utility conflicts are expected in this area.

HST cabling will pass over the structure in the ducting below the walkway areas.

3.11 Hydrological Issues

No hydrological issues are known to affect the bridge location.

3.12 Noise Mitigation and Acoustic Treatment

The bridge location is in the heart of a busy urban area so there may be noise-sensitive receptors in the locality. The bridge will also be adjacent to the site of the new station, which is likely to involve the construction of acoustic screening.

No provision for acoustic screens has been made on the structure, but the details of the parapets will allow noise screening to be added if required.

3.13 Compliance with Systemwide Bridge Aesthetics Features

No specific guidance has been given for structures of this type; however, the structure has been detailed to follow the style of the main viaduct girders to give continuity of appearance. This includes using curved fillets between the deck and the cantilever edges.

3.14 Details of the Geotechnical Parameters Used for Design

See Appendix A.

4.0 Tulare Street Overpass

The HST structure is not a nonstandard or complex structure, and its design has not been developed in detail.

At the 15% stage, the concept indicated was to provide a number of standard precast box beams spanning between conventional abutments. Development of the roadway crossing design for the underpass option has indicated an RC deck supported by contiguous bored piles as the preferred solution, and this solution would be suitable for the HST structure.

4.1 Crossing of the Union Pacific Railroad's Right-of-Way at Tulare Street

In central Fresno the HST alignment is approximately at ground level. In the Tulare Street and Fresno Street area it is planned to construct a new station that will have at-grade access. Because of this, all road crossings that are to remain open must be reconstructed to go either over or under the HST tracks.

The project team has studied overcrossing and undercrossing options at Tulare Street because there was no clear preferred solution. It is expected that a decision on the option to develop will be made by FRA and the HST Authority at a later date.

The overpass option at Tulare Street (i.e., HST going over Tulare Street) requires the construction of a new HST overpass and would require a new bridge to be constructed on the UPRR lines adjacent to the HST right-of-way. Currently, there is no bridge at this location and Tulare Street crosses by means of an at-grade, barrier-controlled crossing.

This bridge is classified as a secondary structure by the HST and would not normally be considered in this report. However, as the construction if this bridge has a significant impact on the main contract, some study has been undertaken to investigate this impact.

It is understood by the JV that UPRR has not been consulted directly on the need for a crossing at Tulare Street thus far, but its "Guidelines for Railroad Grade Separation Projects" state that its most desirable crossing type is an overcrossing (a bridge over their tracks). Furthermore they require applicants to justify the use of an underpass structure "in detail," implying greater detail than would be required for an overcrossing.

The proposed road alignment developed for the 15% design of the Tulare Street overpass ties into the local streets before H Street. The maximum headroom that could be provided at Tulare Street UPRR bridge with this alignment is approximately 15 feet (4.572m), assuming an allowance of approximately 5 feet (1.524m) from Top of Rail for track bed and superstructure thickness. This is less than the required 16 feet 6 inches (5.029m), but is the best that could be achieved, assuming the UPRR would wish the entire width of its right-of-way to be decked out.

The "Guidelines for Railroad Grade Separation Projects" published by UPRR and the BNSF Railway gives guidance on what they may consider as an acceptable undercrossing form.

4.2 Guidelines for Railroad Grade Separation Projects

The UPRR Guidelines state a number of preferences relating to undercrossings that may affect Tulare Street:

- The crossing shall be executed in such a way that the railway tracks remain in service.

- There shall be no interruption to the railroads operation during construction.
- Anything other than the structure types they list are discouraged.
- The types listed by order of preference are
- Rolled beams with deck plate
- Steel plate girders with plate deck
- Rolled beam with concrete deck
- Steel plate girder with concrete deck
- Railroad standard PC double box beams
- Prestressed PC concrete box beams
- Prestressed PC AASHTO beams
- Steel through plate girders with steel deck (but only if all others listed are precluded)
- Cast-in-place superstructures are unacceptable.
- Required vertical clearances are
- 16’ 6” for steel superstructure with beams under (five per track)
- 17’ 6” for concrete superstructure or steel through plate girders with bolted bottom flanges
- 20’ 0” for steel through plate girders without bolted bottom flanges
- Provision may be required for adding tracks and maintenance roads in the future.
- Preferred skew angle zero degrees. Minimum angle between track and supports is 75 degrees for concrete and 60 degrees for steel superstructures if unavoidable.
- Deck width to be based on accommodating a 20-foot track spacing regardless of actual track spacing.

The 15% design assumed that the bridge deck would be required over the full width of the UPRR right-of-way. If this is what UPRR requires, then none of the deck types and vertical clearance combinations listed could be accommodated with the highway vertical alignment that has been designed. The most appropriate bridge types appear to be

- Steel plate girder with steel deck (2)
- Steel through girder (8)

The former is preferable to UPRR and has the least required clearance of 16 feet 6 inches (5.029m), but the beams will be relatively deep, of the order of 4 to 5 feet (1.219m to 1.524m), and would require multiple bearings and a continuous abutment support beam. The latter is UPRR’s least preferred type and has a larger required vertical clearance, but has the advantage of moving the load-carrying component to the side of the track and therefore to a higher level. This means that only the depth of the cross beams need be considered for clearance calculations. However, a through girder structure could not economically support the full width of the UPRR right-of-way.

4.3 Reduced-Width Bridge Decks

The current UPRR corridor in this area supports only two tracks, so it would be possible to bridge Tulare Street with a two-track through bridge that would be approximately 46 feet (13.411m) in width. This is narrower than the full width of the right-of-way but if acceptable to UPRR, it would mean that the point at which headroom was measured would be farther into the undercrossing and the achievable headroom would be greater.

Three types of bridge deck were investigated. A through girder, a multiple beam deck and a precast concrete beam deck. Two can fully satisfy the UPRR vertical clearances.

- A multiple steel beam deck (No 2 on the preference list)
- A steel through girder deck (No 8 on the preference list)

- A precast concrete beam deck

Figures 4.5-1, 4.5-2, and 4.5-3 show the typical cross section for each of the three deck forms investigated.

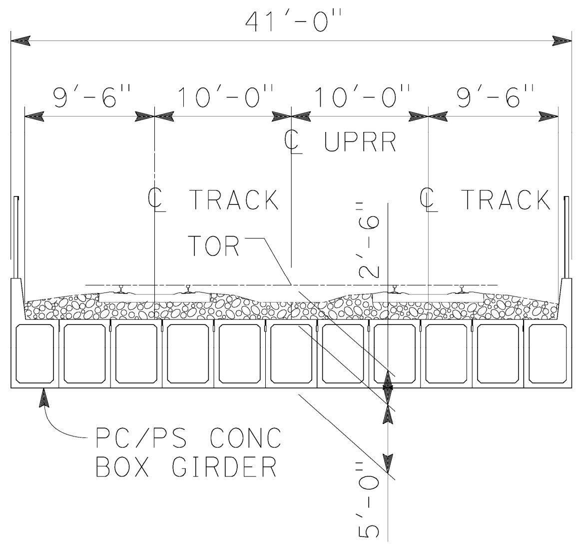


Figure 4.5-1

Precast Concrete Box Girder Deck

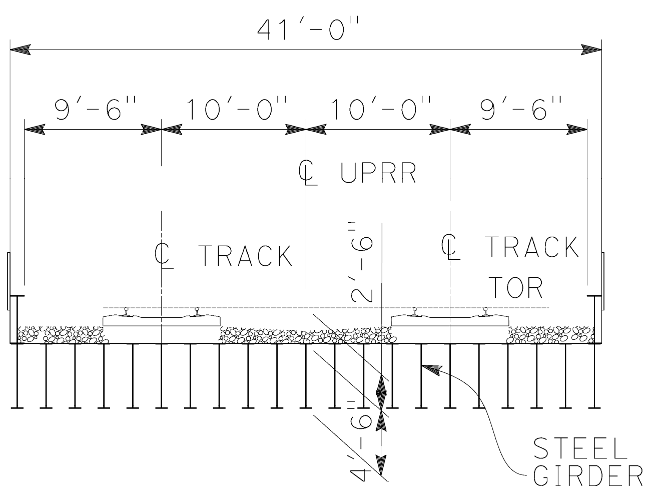


Figure 4.5-2
Multiple Steel Beam Deck

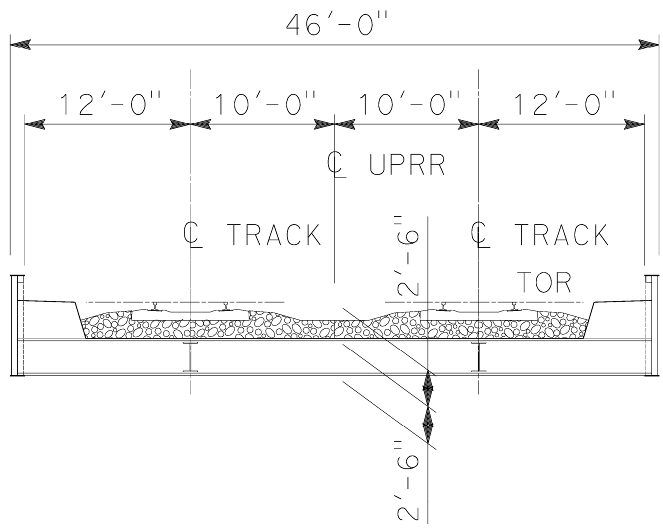


Figure 4.5-3
Steel Through Girder Deck

4.4 Provision of a Shoofly Track

Any shoofly track constructed would have to meet the standards of Union Pacific Railroad. A key requirement would be that the shoofly must not impose any speed restriction on their mainline. The current maximum speed on the mainline is 79mph.

If providing a 79mph shoofly for Tulare and Ventura streets, there would be no opportunity to return to the UP mainline before Fresno St. This would require the use of the new Fresno St bridge due to be constructed for the CHSR mainline.

After passing over Fresno Street on the CHSR mainline alignment, the shoofly should remain on the CHSR alignment, crossing Stanislaus and Tuolumne Streets, before tying into the existing UPRR alignment south of the SR180 overbridge and the spur leading to the SJVRR.

Before construction of the shoofly and switching of the rail traffic, the following would need to be constructed:

- The new Fresno Street underpass
- The western part of the Tulare Street underpass
- The western part of the Ventura Street underpass
- Stanislaus St bridge
- Tuolumne St bridge

Divisadero Street, Kern Street and Mono Street would need to be closed

The road closures of Fresno Street, Tulare Street, Ventura Avenue, Stanislaus Street and Tuolumne Street would need to be coordinated with the City of Fresno and staged to ensure a minimum level of connectivity across the UP tracks.

Although Kern Street and Mono Street would be closed in the permanent case, they may be required as temporary diversion routes during the closures of Tulare Street and Ventura Avenue

respectively. If these were used while the shoofly was in operation, temporary grade crossings would be required, requiring interface with the CPUC.

A connection from the shoofly would be required for the SJVR leading south. This would need a crossover followed by a turnout to their tracks. Changes to the UPRR signaling system associated with the new switches and grade crossings would also have to be phased carefully and could lead to further delays to the overall program.

4.5 Signaling

No data is available regarding the current UPRR signaling system. The shoofly described above would affect the signaling system in order to provide control of the turnouts and switches required for the SJVR operations.

4.6 Possible Method for Avoiding a Shoofly Track

The UPRR guidelines clearly require shoofly tracks to be built so that UPRR train operations are not impacted. However, in a number of places the document suggests that in exceptional circumstances other arrangements may be approved. Connecting the tie-ins of a shoofly inevitably means some disruption, but this could be done overnight to reduce impact. In addition, on completion there would be further disruption to remove the shoofly and reinstate the track.

It may be that another potentially less disruptive method of bridge construction would be acceptable to the UPRR. For example, the following construction sequence is possible:

- Install large-diameter piles for bridge foundations to either side of the tracks (and possibly one between tracks) outside of normal operational hours.
- Construct piles for the retaining walls of the underpass up to the main bridge piles.
- Install way beams on the tracks to provide support to bridge over small excavations. (In this context the way beams are longitudinally aligned, small section, stiff, temporary beams that can be clamped to the ties from above to carry the vertical loads from the trains to supporting areas adjacent to a localized excavation.)
- Using mini-excavators, excavate beneath tracks and construct bearing shelf beam on top of the main piles and retaining wall piles.
- Install prefabricated “through girder” bridge onto bearing beam, including the track bed ballast, ties, and rails.
- During an overnight closure of the UPRR tracks, cut existing rails and remove, lift, or slide bridge into new position and reconnect rails. Re-tamp ballast and reopen to traffic. It is assumed that signaling cables are not affected by this operation and can be repositioned later if necessary.
- Subsequently, excavate underpass beneath bridge placing horizontal steel tube shoring between main piles from side as excavation proceeds to give completed retaining wall.

The form of bridge proposed for this operation is a through girder deck, the least preferred of UPRR’s list above, because it is fabricated as a single unit that only requires four bearings. This makes it quick and easy to slide or crane into position. The precast beam option cannot be constructed as a single unit and so would require more bearings and more time for installation. The steel beam deck could be fabricated as a single unit but it still requires multiple bearings, which add to the time required for installation.

4.7 Summary

Tables 4.7-1 and 4.7-2 summarize the original and currently proposed design with the current road alignment design.

Table 4.7-1
Full UPRR Right-of-Way on Bridge

Deck Form	Order of UPRR Preference	Clearance Required	Min Clearance Achieved
Concrete Box Girder Deck	5 or 6	17'- 6"	14'- 0"
Multiple Steel Beam Deck	2	16'- 6"	15'- 0"

Table 4.7-2
Reduced Width Bridge

Deck Form	Order of UPRR Preference	Clearance Required	Min Clearance Achieved
Through Girder Deck @ 46-foot width (with bolted cross girders)	8	16'- 6"	17'- 3"
Concrete Box Girder Deck @ 41-foot width	5 or 6	17'- 6"	15'- 10"
Multiple Steel Beam Deck @ 41-foot width	2	16'- 6"	16'- 6"

The current Package 1B procurement stage drawings reflect the third type from Table 4.7-2, a multiple steel beam deck.

Appendix A – Geotechnical Design Memorandum

APPENDIX A

Fresno to Bakersfield
Package 1A and 1B

Geotechnical Design Memorandum
for
Nonstandard and Complex Structures

Prepared by:

URS/HMM/Arup Joint Venture

December 2011

Table of Contents

Tables

Figures

ABBREVIATIONS / ACRONYMS

AASHTO	American Association of State Highway and Transportation Officials
ASD	Allowable Strength Design
BGL	below ground level
Caltrans	California Department of Transportation
CBC	California Building Code
CBDS	California Bridge Design Specifications
CDMG	California Division of Mines and Geology
CGS	California Geological Survey
CIDH	cast-in-drilled-hole
HST	California High-Speed Train Project
deg	degrees
DWR	California Department of Water Resources
EL	elevation
EMT	California High-Speed Train Engineering Management Team
ESS	Excavation Support System
FOS	factor of safety
g	gravity
GPS	Global Positioning System
GWL	groundwater level
HMM	Hatch Mott MacDonald
HST	High-Speed Train
in	inches
JV	HMM/URS/ARUP Joint Venture
K	USDA Soil Erodibility Factor
ksf	Kips per Square Foot
LL	lower limit
LRFD	Load and Resistance Factor Design
MCL	Maximum Considered Earthquake
mi	miles
mm	millimeters
M_W	Moment Magnitude
$(N_1)_{60}$	Standard Penetration N-Values Corrected for Hammer Energy, Overburden Pressure, and Field Procedures
N_{60}	Standard Penetration N-Values Corrected for Hammer Energy
NAD27	1927 North American Datum
NAVD88	1988 North American Vertical Datum
NA	not applicable
NCL	Non-Collapse Performance Level
NRCS	Natural Resources Conservation Service
OBE	Operating Basis Earthquake
pcf	pounds per cubic foot
pci	pounds per cubic inch
PGA	peak ground acceleration
PMT	California High-Speed Train Project Management Team
SJV	San Joaquin Valley
SPT	Standard Penetration Test
SPT N	Standard Penetration Test Blow Count
Sta	Station
T	Period
TM	Technical Memorandum
UBC	Uniform Building Code

UL	upper limit
USGS	United States Geological Survey
USDA	United States Department of Agriculture
$(V_S)_{30}$	Average Shear Wave Velocity in the upper 30 meters of soil
yr	year

1.0 Scope

This geotechnical design memorandum addresses the following nonstandard and complex structures in Package 1A (Sta. 10806+00 to Sta. 10970+00) and 1B (Sta. 10970+00 to Sta. 11030+00) in Fresno, California:

- Package 1A structures include the Fresno Grade Separation
- Package 1B structures include the Fresno Street Overpass

Refer to the Section 1.0 of the Trenches, Bridge, and Elevated Structures Report for definitions and locations of nonstandard and complex structures.

2.0 Physiography and Geologic Setting

2.1 Physiography

The California High-Speed Train (HST) Fresno-to-Bakersfield alignment is located in the south portion of the Great Valley Geomorphic Valley (commonly referred to as the San Joaquin Valley), as shown in Figure 2.1-1. The topography of the Great Valley is relatively flat; it is bordered by the Pacific Coast Range to the west, the Klamath Mountains and Cascade Range to the north, the Sierra Nevada to the east, and San Emigdio and Tehachapi mountains to the south.

The general topography of the area surrounding the Fresno subsection comprises low relief terrain at an elevation between about 200 feet and 400 feet above sea level (ASL). However, the topography along the study area is generally between 285 and 295 feet ASL. Superimposed upon this large-scale, relatively flat topography is a localized topography caused by recent incisions of river systems. The subsequent topography comprises short steep river/stream banks with channels at lower elevations relative to the surrounding areas. These channel bottoms range between wide, relatively flat-bottomed (with occasional rounded natural levees) or narrow gully-type valleys, depending on their age and the amount of flow.

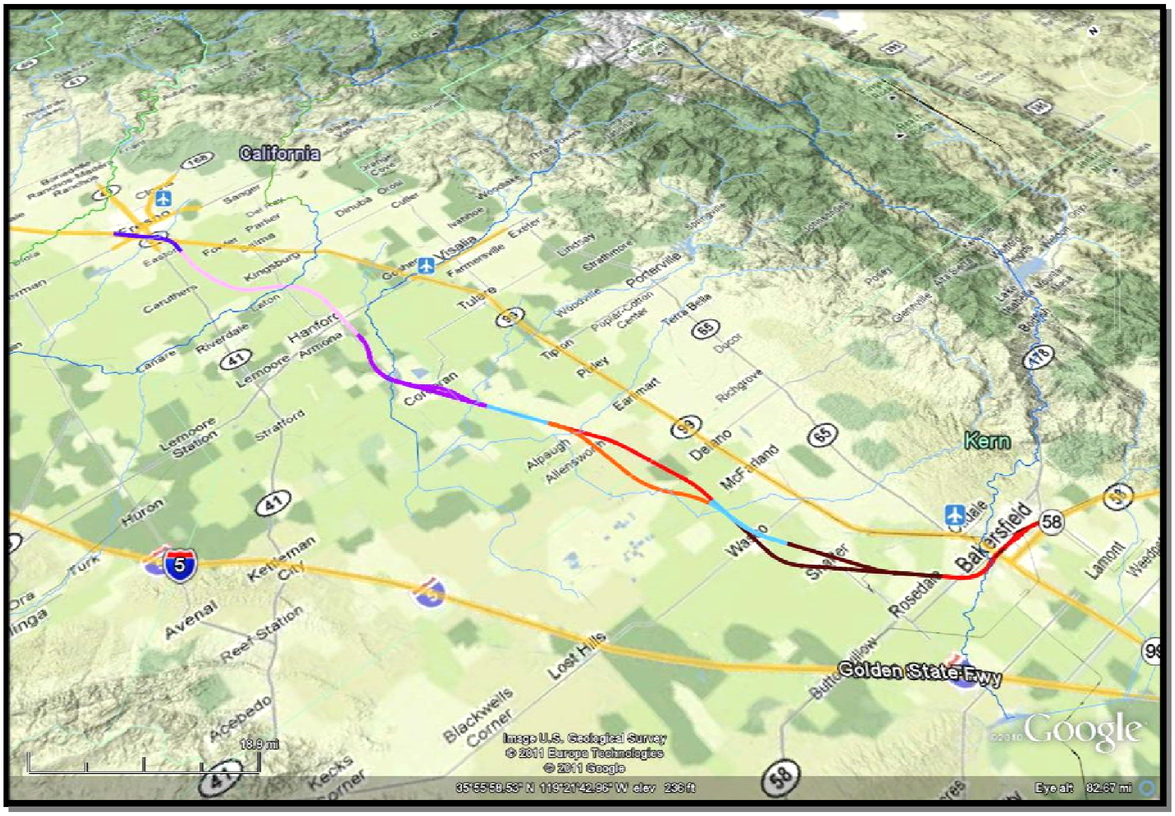


Figure 2.1-1
General Study Area Physiography and Topography (© 2011 Google Inc., 2011)

2.2 Geologic Setting

The San Joaquin Valley (SJV) comprises the southern part of the 450-mile long Great Valley of California and is an asymmetric structural trough that is filled with prism sediments up to 30,000 feet thick. It formed the southern part of an extensive forearc basin that evolved during the Cenozoic into today's hybrid intermontane basin.

The SJV evolved through the gradual restriction of the marine basin due to uplift and emergence of the northern part in the late Paleogene, closing off of the western outlets in the Neogene, and finally the sedimentary infilling in the Neogene and Quaternary. These sediments rest on crystalline basement rocks of the southwestward-tilted Sierran block. Figure 2.2-1 shows a cross-sectional view of these deposits.

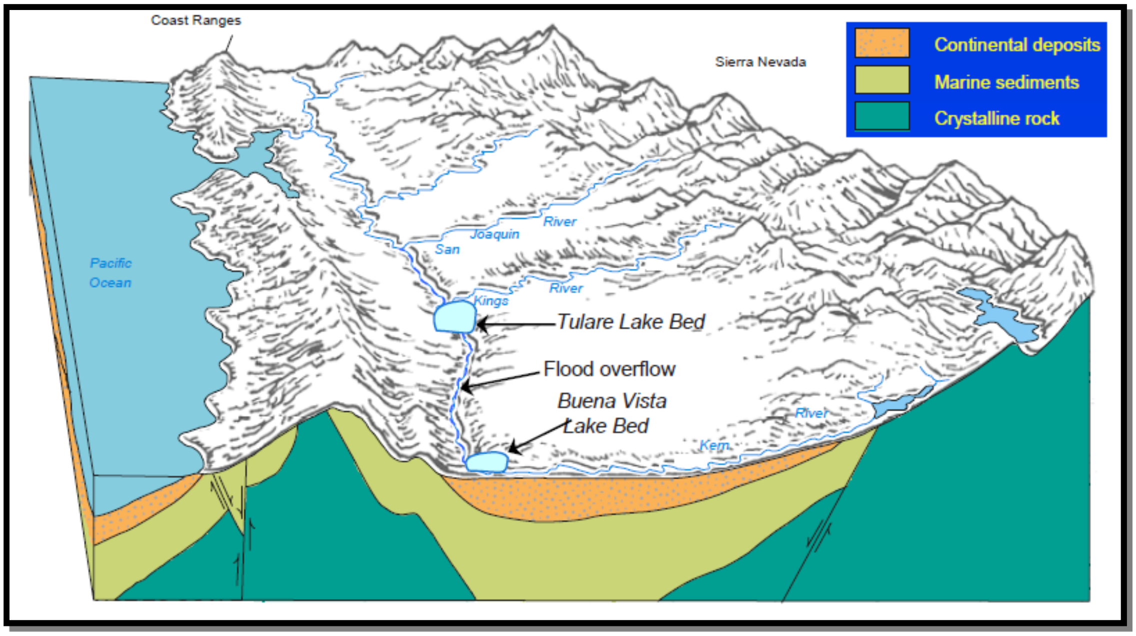


Figure 2.2-1
The Great Valley Geomorphic Province (Page, 1986)

The HST alignment traverses recent alluvial fan deposits (Qf-Modesto Formation) and older Pleistocene nonmarine sedimentary deposits (Qc-Riverbank Formation) as shown on Figure 2.2-2. These deposits originated from stream channels emanating from the foothills east of Fresno. The more recent alluvial fan deposits consist primarily of a mixture of clay, silt, and sand. The older nonmarine alluvium consists primarily of a mixture of slightly consolidated clay, silt, sand, and gravel. The older alluvium is usually situated at a higher elevation and typically exhibits dissected, channelized topography (URS/HMM/Arup, 2011). These deposits may also form a succession of terraces that vary in age. Within the Fresno city limits, artificial fill of various compositions may exist in areas where the alternative HST alignments cross.

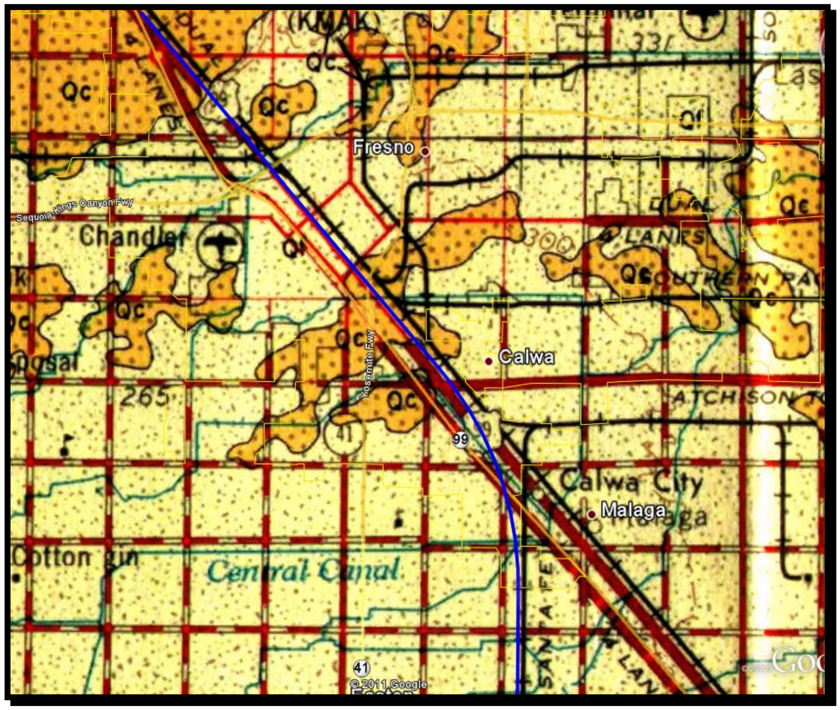


Figure 2.2-2
Local Geology Along the Study Area (Jenkins, 1965)

3.0 Seismic Setting

The project area is located within a relatively seismically quiescent region between two areas of documented tectonic activity, the Coast Ranges-Sierran Block boundary zone to the east and the Pacific Coast Ranges boundary zone to the west.

The Coast Ranges-Sierran Block, which follows the physiographic boundary between the Coast Ranges and Great Valley geomorphic provinces, contains potentially active blind thrust faults (Unruh and Moores, 1992). Based on the size of historical events and on the inferred subsection of the boundary zone, these blind thrust faults are capable of producing moderate to large earthquakes. The Pacific Coast Ranges contain many active faults that are associated with the northwest-trending San Andreas Fault System (Jennings, 1994), which is the principal tectonic element of the North American-Pacific plate boundary in California.

In the SJV, seismic slip is partitioned onto subsidiary structures, such as the San Andreas, Garlock, and Coalinga faults, which are distributed across the Great Valley geomorphic province.

3.1 Faults and Seismicity

There are no known active faults crossing or within close proximity to the alignment along the study area. Accordingly, there are also no designated Alquist-Priolo earthquake fault zones within the study area. The San Andreas Fault has the highest slip rate and is the most seismically active of any fault near the HST alignment. However, other earthquake sources are capable of producing large magnitude earthquakes near the HST alignment. Active and potentially active faults have been mapped in the project vicinity. A more comprehensive list of known faults within the project area is presented in Table 3.1-1.

Table 3.1-1
Fault Characteristics (USGS, 2006)

Fault Name	Fault Type	Moment Magnitude (M_w)	Slip Rate (mm/yr)	Distance & Bearing to Alignment
San Andreas	RL/SS	7.4	20-35	65 miles southwest
Great Valley (Segments 10–14)[1]	BT	6.5	1.5	50 miles southwest
Ortigalita	RL/SS	7.1	0.5 to 1.5	65 miles west
San Joaquin	R	6.9	-	57 miles west
O'Neill	R	6.7	-	58 miles west
Nunez	-	-	-	53 miles southwest
Foothills	N	6.5	0.1	90 miles northwest
Round Valley/Hilton Creek	N	7.0/6.7	1	80 miles northeast
Clovis[2]	-	-	-	12 miles east

[1] Caltrans (1996)
[2] California Geological Survey (2010)
N = normal, BT = blind thrust, R = reverse, RL = right lateral, SS = strike-slip

3.2 Seismic Design Criteria

The system performance criteria approach uses design earthquakes to which HST facilities will be designed. As more devastating earthquakes have a lower probability of occurrence, a probabilistic approach to defining earthquake hazard is used in engineering design. A "return period" identifies the expected rate of exceedance of a given ground motion level. In certain cases, deterministic methods are used to evaluate specific earthquakes that are estimated to produce the most severe ground motion.

3.2.1 Design Earthquakes

For the Fresno portion of the HST alignment, the two design-level earthquakes are defined as follows, in accordance with Technical Memorandum (TM) 2.10.4:

- **Maximum Considered Earthquake (MCE)**: Ground motions corresponding to greater of (1) a probabilistic spectrum based upon a 10% probability of exceedance in 100 years (i.e., a return period of 950 years with 5% damping) and (2) a deterministic spectrum based upon the largest median response resulting from the maximum rupture (corresponding to M_w) of any fault in the vicinity of the structure.

- **Operating Basis Earthquake (OBE)**: Ground motions corresponding to a probabilistic spectrum based upon an 86% probability of exceedance in 100 years (i.e., a return period of 50 years with 5% damping).

3.2.2 Performance Levels

At 30% Design, the MCE corresponds to the Non-Collapse Performance Level (NCL). The main objective is to limit structural damage to prevent collapse during and after a MCE. The OBE governs evaluation of the Operability Performance Level (OPL). The primary objective of the OPL limit state evaluations is to verify that the structures respond elastically to the effects of the OBE with no spalling.

3.2.3 Response Spectra and Peak Ground Acceleration

Procedures for defining the seismic design parameters for the HST are defined in TM 2.10.4. The project management team (PMT) and seconded staff from the Regional Consultants developed site-specific, spectrally matched response spectra and peak ground accelerations for the Central Valley alignment between Merced and Bakersfield. The alignment was divided into eight zones based on shear wave velocities published by the United States Geological Survey (USGS) as well as the variations in the calculated ground motion parameters. This section of the alignment falls within Zone 4 of PMT's study area. Table 3.2-1 summarizes seismic design parameters provided by PMT for 30% Design.

Table 3.2-1
30% Design Seismic Parameters

Seismic Parameter	OBE	MCE
Peak Ground Acceleration (g)	0.08	0.25
Moment Magnitude (M_w)	6.7–7.9	7.1–7.9

The PMT also developed spectrally matched acceleration response spectra for 30% Design for Zone 4 (see Figure 3.2-1). Figure 3.2-1 shows design response spectra for both vertical and horizontal ground motions. Peak Ground Accelerations (PGAs) in Table 3.2-1 were taken as the spectral acceleration at the period (T) of 0.01 seconds.

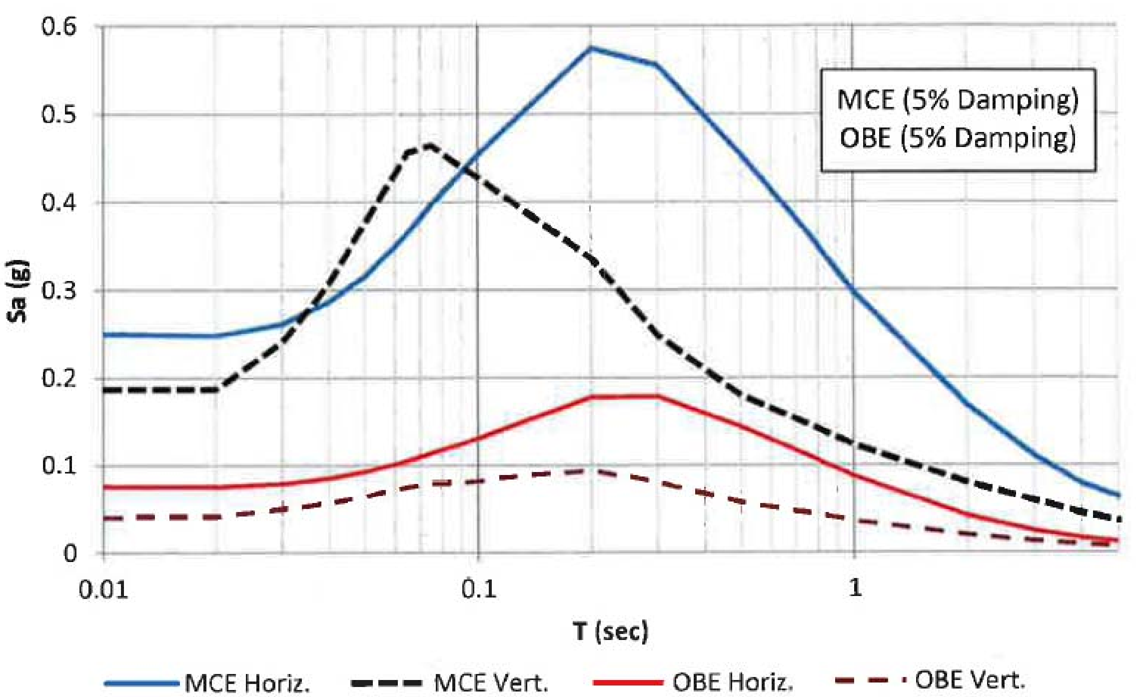


Figure 3.2-1
Design Response Spectra (SC Solutions, 2011)

4.0 Geologic and Seismic Hazards

Table 4.0-1 provides a summary of the geologic and seismic hazards pertinent to the study area. A more detailed summary of geologic and seismic hazards is provided in the Fresno to Bakersfield Geologic and Seismic Hazard Report (URS/HMM/Arup 2011).

Table 4.0-1
Summary of Geologic and Seismic Hazards

Hazard	Risk Level	Location	Reason
Surface Ground Rupture	NA	Entire alignment	No Fault Crossing
Seismically Induced Ground Deformation	Low	Entire alignment	Liquefaction, lateral spreading, settlement
Seismically Induced Flooding	Moderate	Portion of alignment	Dam failure
Volcanic Hazards	Low	Entire alignment	Ash deposition
Land Subsidence	Low	Entire alignment	-
Areas of Difficult Excavation	Moderate	Sta. 100+00 to Sta. 380+00	Hardpan
Unstable Soils	Low–Moderate	Sta. 100+00 to Sta. 380+00	Expansive soil
Soil Corrosivity	Moderate–High	Entire alignment	-
Erodible Soils	Moderate	Entire alignment	Soil Erodibility, K, Factor ~ 0.32
Seasonal Flooding	Moderate–High	Portion of alignment	-
Slope Instability	Low	Entire alignment	Flat Terrain
Hazardous Minerals	Low	Entire alignment	Radon
Abandoned Mines and Karst	NA	Entire alignment	-

The following discusses provided more detail on several of the geologic and seismic hazards with moderate risk levels.

4.1 Difficult Excavation

Areas of difficult excavation are likely to be encountered along the study area. The San Joaquin soil series (W Clinton Avenue to W Belmont Street), which makes up approximately 30% of the study area, has a hardpan present between 12 to 48 inches below the surface (USDA and NRCS, 2008). Hardpan in this area can vary from 4 to 17 inches thick. Similar hardpan may be encountered in the Greenfield series (W Belmont Avenue to Tuolumne Street), which makes up 8% and 4% of the Fresno section, respectively.

This hardpan may impede the excavation of surface material during construction. To mitigate this issue, special equipment may be necessary depending on the characteristics of the layer. A subsurface investigation should be conducted to determine these characteristics, such as the extent and strength of the hardpan.

4.2 Expansion Potential

The expansion potential along the alignment is summarized in Table 4.2-1.The soil along HST Package 1 alignment consists mainly of silty sand, silt, or lean clay and therefore has a low expansive potential. The San Joaquin soil series, however, has a low to moderate expansive potential (USDA Soil Survey definition) to 16 inches and a moderate expansive potential (USDA Soil Survey definition) from 16 to 30 inches below the surface (Huntington, 1971). The plasticity of these soils ranges from nonplastic to 30. The San Joaquin soil series makes up approximately 30% of the study area and is generally found between W Clinton Avenue and W Belmont Avenue.

Table 4.2-1
Extent of Expansive Soils

Location	Expansive Potential	Depth (in)
W Clinton Ave to W Belmont Ave	Low–Moderate	0–16
W Clinton Ave to W Belmont Ave	Moderate	16–30

4.3 Corrosion Potential

The soils along the study area have a moderate to high risk of corrosion for uncoated steel and a low to moderate risk of corrosion of concrete (USDA and NRCS, 2008). The San Joaquin soil series, which makes up approximately 30% of the study area and is generally found between W Clinton Avenue and W Belmont Avenue, has a high risk of corrosion for uncoated steel and a moderate risk of corrosion for concrete. This San Joaquin soil is generally composed of a combination of silty sand, silty clay, and lean clay and has a soil pH ranging from 5.6 to 6.5 near the surface and 6.1 to 7.3 to a depth of 5 feet, with zero soil salinity.

The Hanford soil series, which makes up approximately 24% of the study area and is generally found between W Franklin Avenue and Fresno Street, has a moderate risk of corrosion for uncoated steel and a low risk of corrosion for concrete. The Hanford soil series is composed of a combination of silty sand and silt, and has a soil pH from 6.1 to 7.3 and zero soil salinity.

Table 4.3-1 provides a description of the corrosivity risk level within the study area.

Table 4.3-1
Risk of Corrosion for Uncoated Steel and Concrete

Location	Risk of Corrosion Uncoated Steel	Risk of Corrosion Concrete
W Clinton Ave to Ventura Ave	Moderate–High	Low–Moderate

4.4 Hydrocompaction and Collapse Potential

Detailed evaluations of hydrocompaction of soils along the study area could not be found, likely because the HST alignment is located outside areas in the SJV that have historically experienced hydrocompaction. Moreover, outside of the urban corridors, the alignment traverses through heavily irrigated farmlands. Although none has been recorded, any hydrocompaction through irrigated farmlands has likely long since been exhausted.

Laboratory test data included from historical geotechnical reports within the area of study (Technicon, 2004) suggest very low dry densities, which are typically indicative of collapse potential. Data indicates a collapse potential of up to 2 to 4% upon wetting under the 2,000 pounds per square foot load increment.

4.5 Subsidence

No information is available on historical land subsidence within the study area. The area may have experienced land subsidence in the early 1900s when it was prevalent in the SJV. However, no significant land subsidence is known to have occurred in the last 50 years as a result of land development, water resources development, groundwater pumping, or oil drilling. A Global Positioning System (GPS) control network has been established throughout the Plan Area. This control network consists of more than 20 control points that are tied to the High Precision Grid Network using the 1988 North American Vertical Datum (ASL). This control network is utilized to survey existing local benchmarks to monitor subsidence (FID et. al., 2006).

A cursory evaluation of subsidence along the alignment was made by comparison of the current ground surface elevations along the alignment — taken from the Fresno to Bakersfield 30% Record Set Plan & Profile Sheets — to ground surface elevations from Google Earth.

Based on this evaluation, there does not appear to be any subsidence within the study area. Considering the magnitude of groundwater declines within the City of Fresno and the ongoing subsidence in neighboring communities, largely due to groundwater abstraction, the lack of documented subsidence is rather remarkable. However, communities in the SJV that are experiencing substantial groundwater-abstraction-induced subsidence are also underlain by substantial clay deposits including the Corcoran Clay, whereas Fresno is not. Based on past ground response performance, the risk of subsidence within the study area is considered low.

5.0 Historical Geotechnical Data

5.1 Geotechnical Investigations

No site-specific geotechnical investigation was available for the preliminary design of the structures (i.e., trenches, bridges, and elevated structures) in Package 1A and 1B of the Fresno to Bakersfield Section of the HST alignment. The preliminary design summarized in this report is based on historical geotechnical data along the HST study area in Fresno, mainly along State Routes 41, 43, and 99, as well as those of the City of Fresno residential development projects.

Caltrans has completed a large number of projects along the HST study area in Fresno, mainly along Routes 41, 43, and 99. The project dates range from 1953 to 1997. For each project, several boreholes were drilled, logged, and plotted on a cross section.

In total, data is available from about 350 boreholes within 2 miles of the alignment; however, many of the borehole logs offer little detail about the soils beyond the potential depth of the viaduct foundations. The boreholes extend to a maximum depth of 121.8 feet below ground level (BGL), with an average borehole depth of 42 feet BGL.

The City of Fresno provided 14 geotechnical data reports for residential development projects. These reports are dated between 1987 and 2007 and contain 135 borehole logs. The boreholes range in depth from 10 feet to 21.5 feet BGL.

5.2 Stratigraphy

Generally, the stratigraphy within the study area consists of alternating layers of poorly graded sand (SP), silty sand (SM), silt (ML), and silty clay (CL). More specific information, based on historical ground investigations, is presented below.

Where Route 99 and W Clinton Avenue cross the alignment in northern Fresno, the ground surface at time of drilling was around elevation (EL) 295 feet (ASL) (Caltrans 1953a, 1953b, 1990). Standard Penetration Test (SPT) N-values (blow counts) in granular soils above EL 265 feet (between 0 to 30 feet deep) generally range from 4 to 30, which corresponds to loose to medium dense soil. Below EL 265 feet, N-values are generally greater than 30, corresponding to dense to very dense soil. However, several boreholes show high N-values near the surface, which may be due to the hardpan. The stratigraphy within the study area can be described as follows:

- Ground level to 15 ft BGL (EL: 285–270 ft ASL) – alternating beds of loose to very dense poorly graded sand, silty sand, and silt (SPT N 4–100) including possible hardpan at shallow depth
- 15–25 ft BGL (EL: 270–260 ft ASL) – beds of stiff to hard silt (SPT N 16–60)
- 25–45 ft BGL (EL: 260–240 ft ASL) – alternating beds of dense to very dense poorly graded sand, silty sand, silt and poorly graded sand with silt (SPT N 31–90)
- 45–70 ft BGL (EL: 240–225 ft ASL) – alternating beds of dense to very dense poorly graded silty sand, stiff to hard silt, and low plasticity silty clay (SPT N 27–170)
- Groundwater was not recorded

Figure 5-2.1 shows the locations of existing geotechnical data from Caltrans borings near where the proposed alignment intersects Route 180.

Figure 5.2-2 shows the variation of SPT blow counts for some of the boreholes at the intersection of W Nielsen Avenue and Route 99 about a half a mile east of the alignment, near the northern end of the Fresno Grade Separation. Figure 5.2-3 shows the variation of SPT blow counts for some of the boreholes between W Nielsen Avenue and Route 180 on Route 99.

Please note that SPT N-values in Figures 5.2-2 and 5.2-3 have been converted to N_{60}. Values greater than 100 have been omitted from the figures.

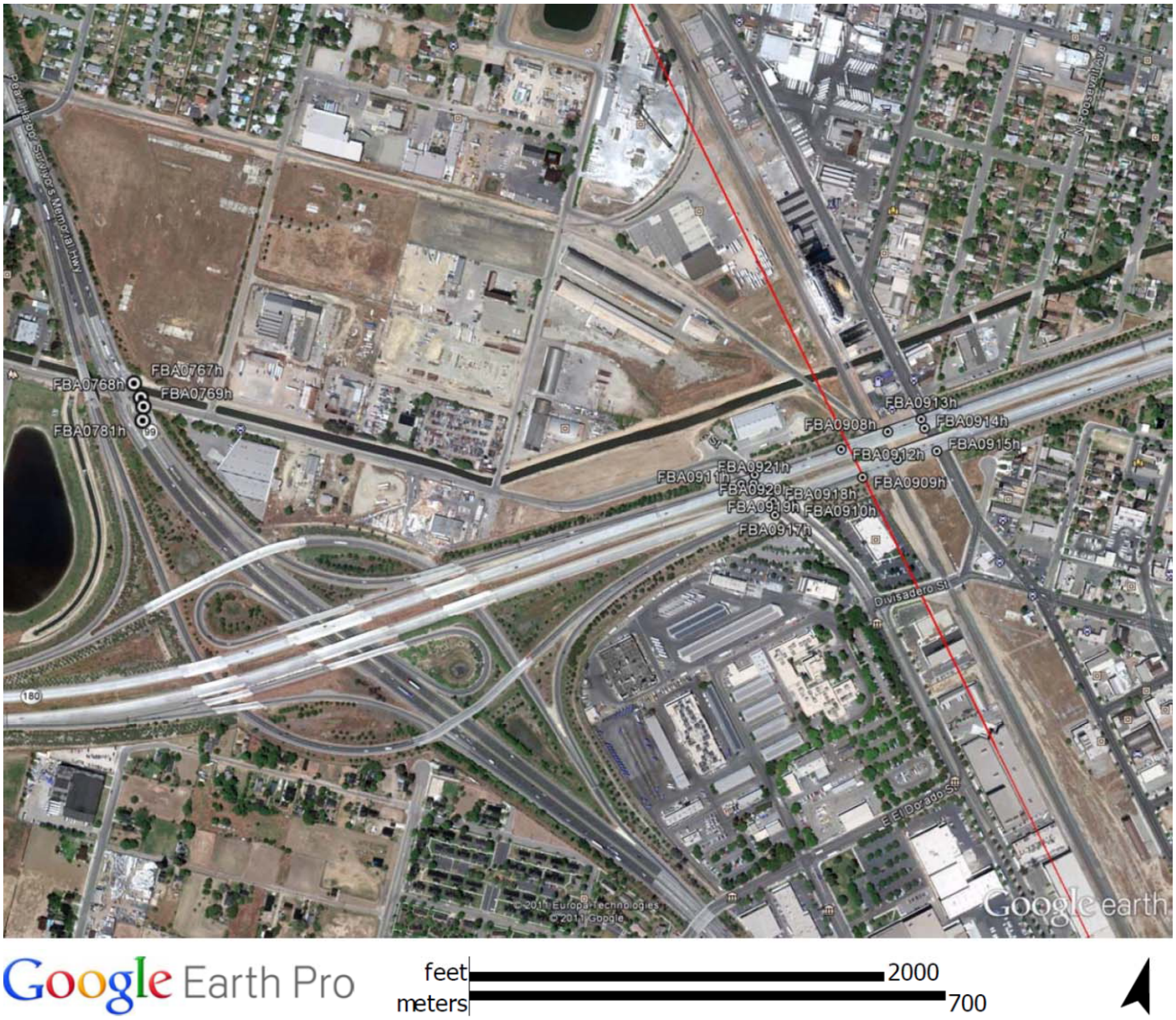


Figure 5.2-1
Map of Existing Geotechnical Data near Route 180 Intersection (© 2011 Google Inc., 2011)

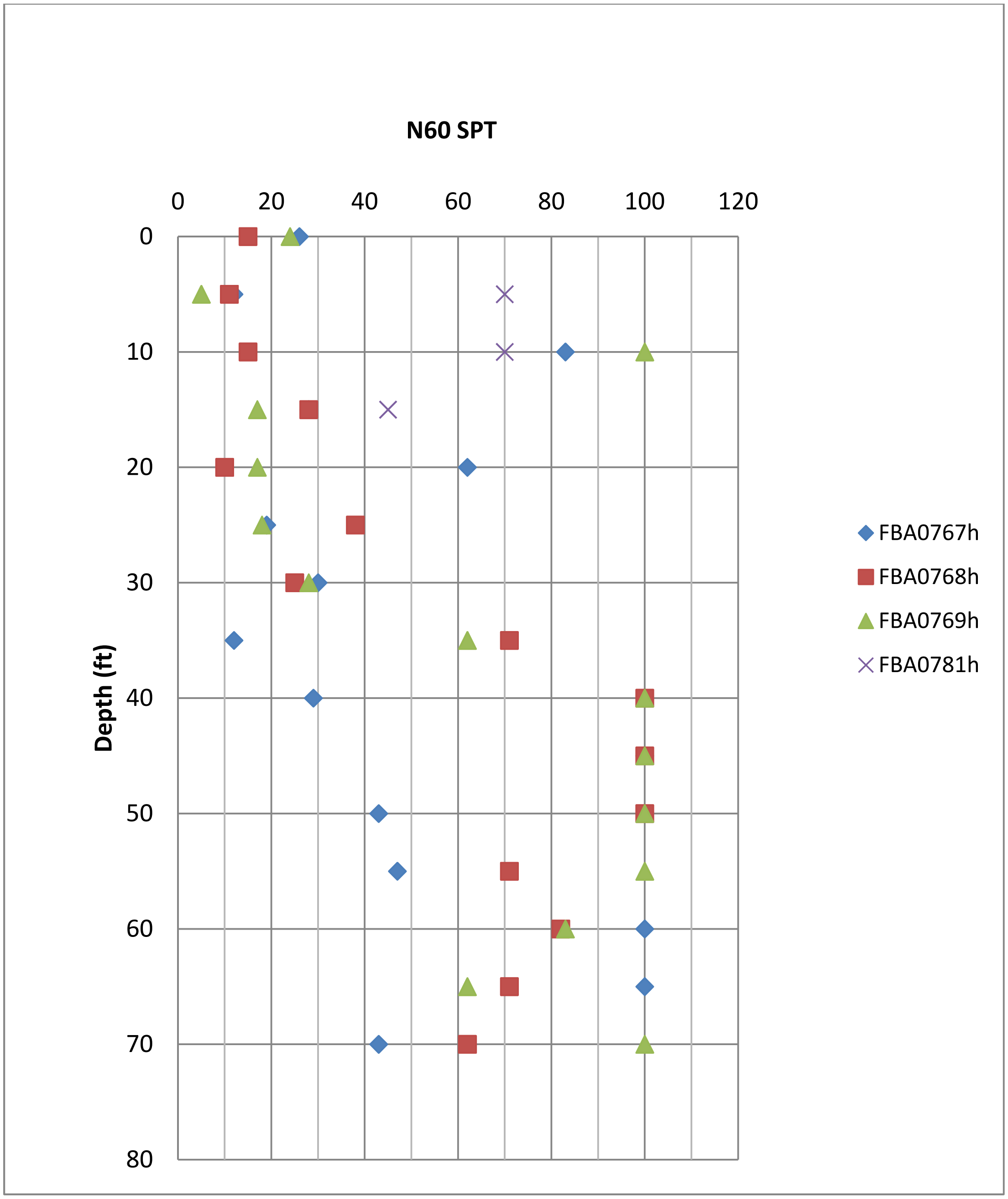


Figure 5.2-2
N_{60} Values at Route 99 and W Nielsen Ave

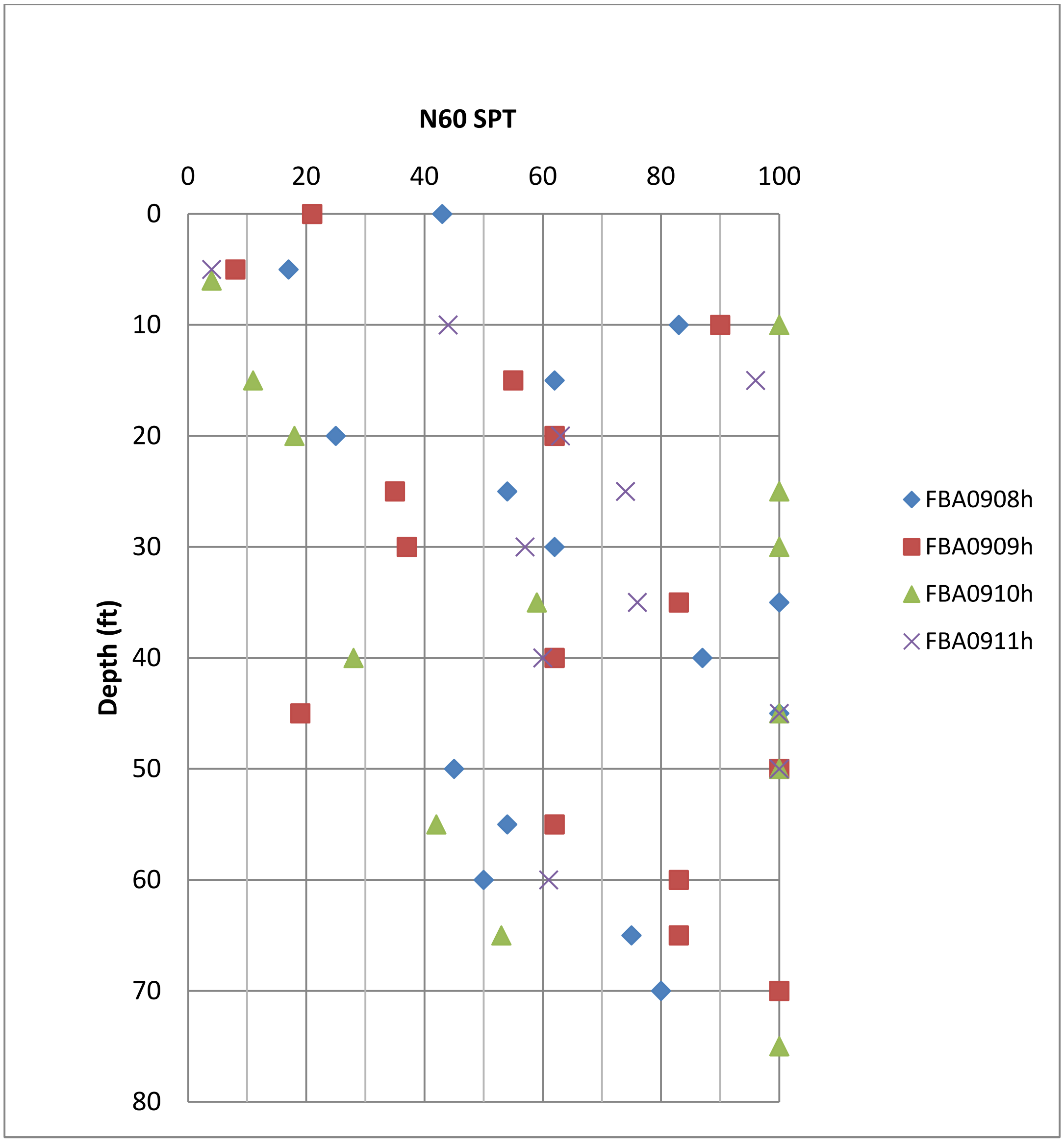


Figure 5.2-3
N_{60} Values at Route 99 between W Nielsen Ave and Route 180

Near where Route 41 crosses the alignment (slightly south of the study area), the ground surface at time of drilling was around EL 285 feet (Caltrans, 1963). N-values vary greatly in this area, ranging from 4 to 100. In the top 10 feet, they tend to be between 4 and 20, which correspond to very loose to medium dense soil. Below 10 feet, density ranges from medium dense to very dense soil. Some boreholes show a clayey silt at approximately EL 245 feet (40 feet). The stratigraphy within this reach of the alignment can be described as follows:

- Ground level to 15 ft BGL (EL: 285–270 ft ASL) – alternating beds of loose to very dense poorly graded sand, silty sand, and silt (SPT N 4–100) including possible hardpan at shallow depth
- 15–25 ft BGL (EL: 270–260 ft ASL) – beds of medium dense to very dense silt (SPT N 16–150)
- 25–45 ft BGL (EL: 260–240 ft ASL) – alternating beds of dense to very dense poorly graded sand, silty sand, silt, and poorly graded sand with silt (SPT N 31–90)

- 45–60 ft BGL (EL: 240–225 ft ASL) – alternating beds of dense to very dense poorly graded silty sand, silt, and low plasticity silty clay (SPT N 27–70)
- Groundwater was not recorded

Figure 5-2.4 shows the locations of existing geotechnical data from Caltrans borings near where the proposed alignment intersects Ventura Avenue.Please note that SPT N-values in Figure 5.2-4 have been converted to N_{60}. Values greater than 100 have been omitted from the figure.

Figure 5.2-5 shows the variation of SPT blow counts with depth for some of the historical boreholes at the intersection of Highway 99 and Ventura Avenue.

Figure 5.2-4
Map of Existing Geotechnical Data Near Proposed Alignment Intersection with Ventura Ave
(© 2011 Google Inc., 2011)

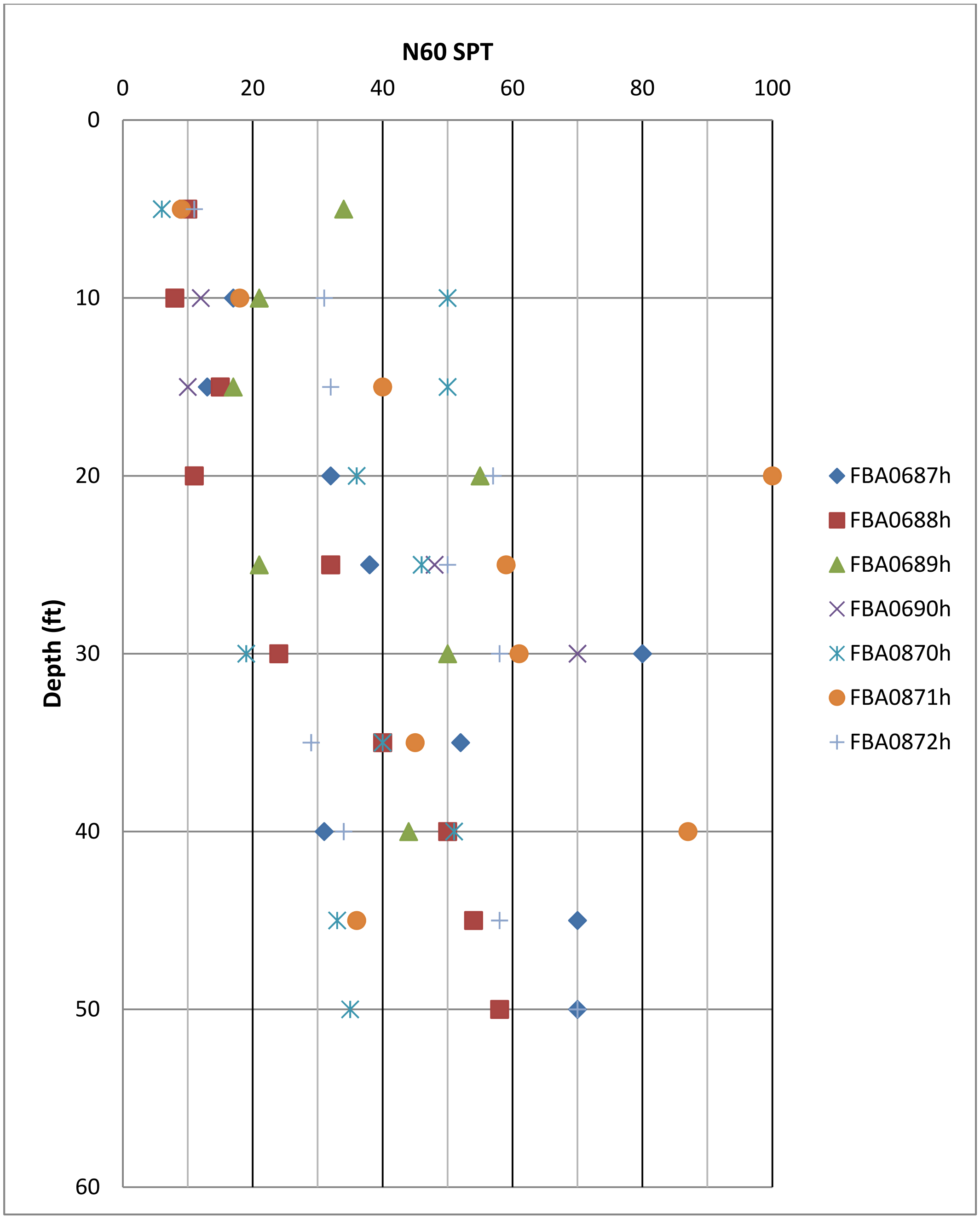


Figure 5.2-5
N_{60} Values at Route 99 and Ventura Ave

5.3 Laboratory Testing

While there is no laboratory data associated with the Caltrans investigations, the City of Fresno geotechnical data reports contain a moderate amount of laboratory data. Since the City of Fresno boreholes only extend to a depth of approximately 20 feet BGL, all of the historical laboratory data is above this level. Table 5.3-1 summarizes the available historical laboratory test data.

Table 5.3-1
Historical Laboratory Testing Data

Test	Minimum	Maximum	Mean	No. of Results
Moisture Content (%)	1.7	33.8	11.3	180
Dry Density (pcf)	77.8	132.2	112.0	154
Fines Content (%)	3.8	99.0	42.2	31
Cohesion (psf)	0	500	300	7
Friction Angle (deg)	26.3	40.0	33.4	9
Void Ratio	0.35	0.88	0.56	26
Peak Shear Strength (ksf)	0.3	2.4	1.4	15
Residual Shear Strength (ksf)	0.7	2.2	1.4	9
Coefficient of Virgin Compression, Cc	0.02	0.06	0.04	7
Coefficient of Recompression, Cr	0.01	0.01	0.01	7
pH	6.4	9.6	8.0	2

5.4 Groundwater Levels

Historically, the groundwater table elevation has fluctuated but has generally experienced a depletion of about 50 feet since the 1960s. Prior to urbanization and agricultural pumping, the groundwater table was within 20 to 30 feet of the ground surface. For the majority of the boreholes done by Caltrans, groundwater was not encountered. However, in October 1959, the groundwater in the vicinity of Route 99 at Central Avenue was between EL 245 feet and EL 264 feet (26.5 to 38 feet BGL). In June 1997, the groundwater in the vicinity of the intersection of Route 99 and Route 180 was between EL 196 feet and EL 203 feet (73.8 to 76.1 feet BGL).

Table 5.4-1 summarizes the historical groundwater levels along the alignment over the past 50 years.

Table 5.4-1
Groundwater Table Depths (ft BGL)

Location	1960–65	1986–88	1999–01	2005	2009–11
Clinton	70	88	98	110	-
Roeding Park	64	84	94	100	-
Ventura Ave	59	71	80	101	125

Figure 5.4-1 shows a hydrograph of historical water well levels in the city of Fresno over the past 80 years. This hydrograph is reasonably consistent with hydrographs of wells along the alignment presented in the HST Fresno-to-Bakersfield Geologic and Seismic Hazard Report (URS/HMM/ARUP, 2011) along the HST alignment, which show a general trend of groundwater depletion within the Fresno city limits.

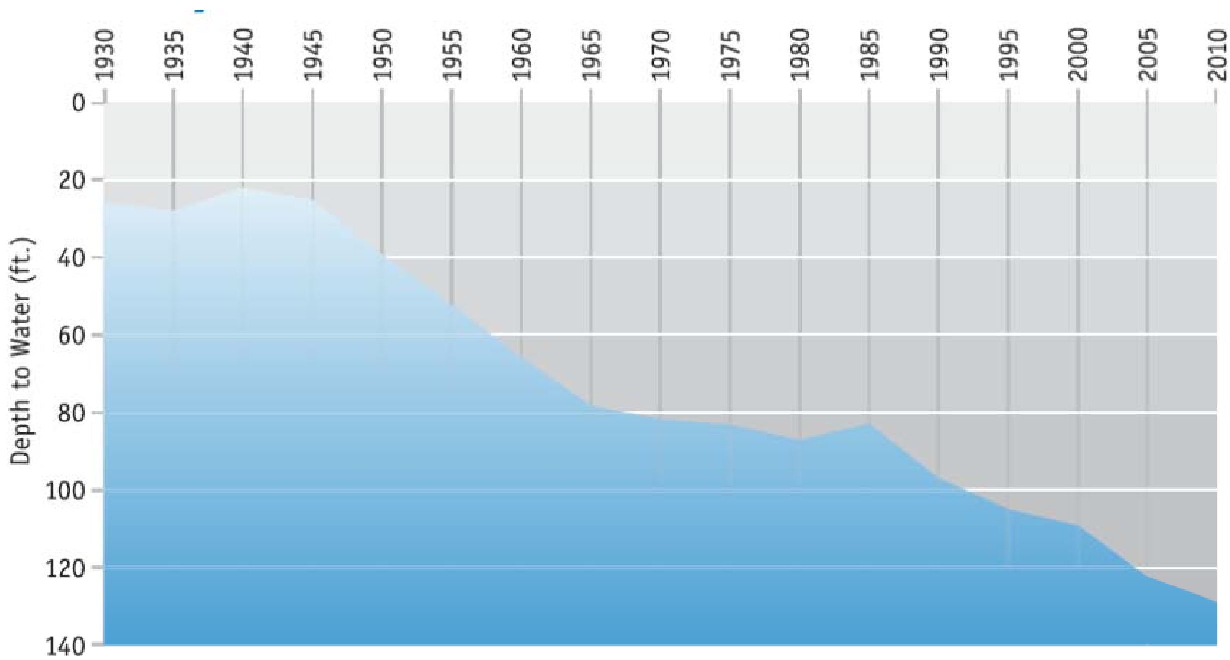


Figure 5.4-1
City of Fresno Historical Water Levels (City of Fresno, 2010)

Current groundwater in the unconfined aquifer is likely to be encountered approximately 120 feet BGL from Clinton to Ventura Avenue, with possible localized mounding in the vicinity of the Dry Creek Canal.

For preliminary design purposes, however, a groundwater table depth of 40 feet is presumed to be reasonable throughout the study area — with the exception of localized water mounding that may occur at water crossings such as Dry Creek Canal, where a design groundwater table of 10 feet is more appropriate.

5.5 Ground Model

For the interpretation of the historical geotechnical data and the geologic setting of the Fresno subsection, a "typical" ground model is assumed for the design of each complex and nonstandard structure. This ground model represents the worst credible geotechnical situation. It is anticipated that 70% of the geotechnical conditions are better than the typical ground model and about 30% of the foundation conditions are worse.

The design parameters used for this section are based on the limited data available from historical geotechnical explorations. These explorations were not intended for final design of the structures necessary for the HST project. Since rock conditions are not indicated in the historical data reviewed, the developed ground model does not contain any rock properties.

The design soil profiles for the Fresno Street HST Overpass are shown in Table 5.5-1.

Table 5.5-1
Design Soil Profile for Fresno Street HST Overpass

Soil Design Parameters	Structural Fill Layer A	Silty Sand Layer B	Silty Sand Layer C
Thickness of layer (ft)	35	25	>70
N-Value Corrected for Hammer Energy, N_{60} (blows/ft)	30	18	50
Friction Angle, ϕ' (deg)	37	35	37
Total Unit Weight, γ (pcf)	125	125	125
Young's Modulus, E (ksf)	980	760	980

The design soil profiles for the north and south ends of the Fresno Grade Separation are shown in Tables 5.5-2 and 5.5-3.

Table 5.5-2
Design Soil Profile for Fresno Grade Separation – North of Sta. 10924 + 00

Soil Design Parameters	Silty Sand Layer A	Silty Sand Layer B
Thickness of layer (ft)	40	>60
N-Value Corrected for Hammer Energy, N_{60} (blows/ft)	20	70
Friction Angle, ϕ' (deg)	35	40
Total Unit Weight, γ (pcf)	125	125
Young's Modulus, E (ksf)	800	1540

Table 5.5-3
Design Soil Profile for Fresno Grade Separation – South of Sta. 10924 + 00

Soil Design Parameters	Silty Sand Layer A	Silty Sand Layer B	Silty Sand Layer C
Thickness of layer (ft)	10	10	>80
N-Value Corrected for Hammer Energy, N_{60} (blows/ft)	20	45	60
Friction Angle, ϕ' (deg)	37	40	39
Total Unit Weight, γ (pcf)	125	125	125
Young's Modulus, E (ksf)	940	1540	1420

6.0 Liquefaction

The liquefaction assessment for the Fresno Package 1 alignment was performed for both the OBE event (return period of 50 years) and the MCE event (return period of 950 years). These calculations were performed based on the historically available borehole Fresno to Bakersfield database.

Soil liquefaction is the loss of shear strength in sandy soils due to an increase in pore pressure during dynamic loading. It is most commonly associated with shallow, loose, saturated deposits of cohesionless soils subjected to strong earthquake shaking, often causing significant damage. The vast majority of liquefaction hazard is associated with sandy soils and silty soils of low plasticity. Cohesive soils are generally not considered susceptible to soil liquefaction. In order to be susceptible to liquefaction, potentially liquefiable soils must be saturated or nearly saturated. In general, the hazards are most severe in the upper 50 feet of the surface, but on a slope near a free face or where deep foundations go beyond that depth, liquefaction potential should be considered at greater depths (CGS, 2008).

As per the Geotechnical Analysis and Design Guidelines (TM 2.9.10), the liquefaction assessment should be performed to a depth of 75 feet according to one of the following methods:

- Youd et al. (2001)
- Seed et al. (2003)
- Idriss and Boulanger (2008) (not used)

For the purpose of this study, the Youd et al. (2001) as well as the Seed et al. (2003) methodologies were implemented. The final results are reported in terms of Factor of Safety (FOS) as per TM 2.9.10 and conclusions are drawn accordingly. Since more than one approach was used in this study, if different methods resulted in values within 20% of each other, the presented FOS values were averaged. Otherwise, the more conservative of the results (lower FOS) was reported. This study provides an SPT-based liquefaction evaluation.

6.1 Methodology

For the 30% Seismic Design stage, two levels of design earthquakes are considered: OBE and MCE.

The liquefaction assessment was based on the historical SPT measurements, and the analysis followed the research and subsequent methodologies described by Youd et al. (2001) and Seed et al. (2003). For each methodology, a minimum required $(N_1)_{60}$ curve to resist liquefaction (or liquefaction triggering curve) was defined with depth. This triggering curve was determined based on the liquefaction curves and the appropriate fines content following the procedures and assumptions of each method. Additionally, for both methods, the measured SPT N-values were corrected to $(N_1)_{60}$ as recommended by Youd et al. and Seed et al. The hammer energy efficiency correction, overburden pressure correction, and other factors are listed below:

$(N_1)_{60} = N.C_N.C_R.C_S.C_B.C_E$
C_N = correction for overburden pressure
C_R = correction for short rod length
C_S = correction for nonstandardized sampler configuration (= 1)
C_B = correction for borehole diameter (= 1.0)
C_E = correction for hammer energy efficiency (= 1.25)

The corrected SPT N data $(N_1)_{60}$ were then plotted on the same graphs with liquefaction triggering curves for comparison.

6.2 Assumptions

The PGA values used for the analyses presented in Table 3.2-1 are based on the 30% Design Ground Motions Report for the HST project performed by SC Solutions (2011). The report assumed an average shallow shear wave velocity $(V_S)_{30}$ of 935 feet per second for the liquefaction analysis within the study area. For the Seed et al. (2003) method, an average shear wave velocity $(V_S)_{40}$ of 935 feet per second was also considered over the top 40 feet of soil.

The earthquake magnitudes for the design earthquakes, based on the 30% Design ground motions drawings provided by SC Solutions, are summarized in Table 3.2-1. A lower limit and an upper limit magnitude were assigned to each design earthquake.

Since the available geotechnical borehole data were sparse and not located along the alignment, soil layers with a uniform total unit weight of 125 pounds per cubic foot were assumed throughout Package 1. Review of the historical borehole data shows extensive existence of silty sand and lean clay in the region. However, since the available quantitative information about the fines content of the soils was very limited, a wide range of fines content was considered in the liquefaction assessment: 5%, 15%, and 35%. This sensitivity analysis demonstrates the effect of fines content on the liquefaction susceptibility.

For the liquefaction assessment two groundwater levels are considered, 10 feet and 40 feet BGL. Based on existing borehole information, a groundwater depth of 40 feet is expected in most locations. However, since the effect of seasonal fluctuations and local perched water could be prevalent throughout the study area, a model groundwater depth of 10 feet was used in the liquefaction assessment as a worst-case scenario.

6.3 Results

A liquefaction assessment was carried out for the Fresno area using two simplified methods based on the assumptions elaborated in the previous section.

The results of this assessment are presented in Figures 6.3-1 to 6.3-8, each based on different assumptions. Figures 6.3-1 and 6.3-2 present the results for Youd et al. (2001) and Seed et al. (2003) methods, respectively, for the case of the OBE and a groundwater level of 10 feet. These figures reveal that there is no liquefaction hazard within the study area under this hazard level.

Figures 6.3-3 and 6.3-4 illustrate the results for Youd et al. (2001) and Seed et al. (2003) methods, respectively, for the case of the OBE and a groundwater level of 40 feet. As expected from the previous cases, liquefaction is not triggered at any depth for either lower- or upper-limit magnitude. For the Seed et al. (2003) method for OBE with lower-limit magnitude (Figures 6.3-2 and 6.3-4), the triggering curves are not shown in the plots. This is because the calculated cyclic shear stresses are small and no corresponding data can be read from the liquefaction triggering curves. Although the liquefaction triggering curves were calculated for different fines content (5%, 15%, 35%), the results suggest that this parameter does not have any effect on the liquefaction susceptibility under the OBE shaking level because of the very small cyclic shear stress ratios induced in the OBE case.

Figures 6.3-5 and 6.3-6 present the results for the case of the MCE with a groundwater depth of 10 feet. Comparing the left (lower-limit magnitude) and the right (upper-limit magnitude) plots in each figure, it becomes apparent that the liquefaction susceptibility increases once larger earthquake magnitudes are applied. More specifically, for the lower-limit magnitude earthquake with groundwater modeled at a depth of 10 feet, a few data points suggest liquefaction potential for fines content of 5%.

These figures show that as the fines content increases, potential for liquefaction decreases. In summary, the liquefaction likelihood for the MCE with a lower-limit level and groundwater level of 10 feet is moderate. This is because most of data points suggesting the potential for liquefaction are single sample points in the boreholes, which does not suggest a continuous layer spatially.

For the upper-limit magnitude earthquake, there are more points showing the potential for liquefaction. The boreholes with potentially liquefiable material under this level of ground shaking are concentrated on the northern and southern part of the alignment within the study area. Again, as the fines content increases, the liquefaction potential decreases. Thus, for greater fines content of 15% or 35%, the liquefaction hazard for northern and southern parts is reduced.

Figures 6.3-7 and 6.3-8 present the results for Youd et al. (2001) and Seed et al. (2003) methods, respectively, for the case of the MCE and a groundwater level of 40 feet. According to these figures, there are very few data points showing the potential for liquefaction in the upper-limit magnitude case. Thus, there is no liquefaction potential for either MCE level when a groundwater depth of 40 feet is assumed.

The observations from these sensitivity analyses highlight the importance of an adequate geotechnical investigation to correctly monitor the level of groundwater. This should reflect the seasonal fluctuations and address the perched water existence. Furthermore, the effect of soil type (e.g., sandy or clayey) will have a remarkable impact because the fines content affects the cyclic behavior of the soils. Liquefaction potential shall be reevaluated in the future as site-specific geotechnical investigation data becomes available.

The liquefaction potential (FOS<1.2) for all boreholes is summarized in Table 6.3-1. The FOS shown corresponds to the average of FOS for data points below a FOS of 1.2. Liquefaction only occurs under the MCE shaking level with a groundwater level of 10 feet and is more sever for the upper-limit earthquake magnitude.

Table 6.3-1
Fresno Liquefaction Evaluation Results

Design Earthquake	Groundwater Depth (ft)	Earthquake Magnitude (M_w)	Fines Content (%)	No. of boreholes with potential liquefaction	Average FOS	Liquefaction potential
OBE	10	6.7 (LL)	5	0	-	No
			15	0	-	No
			35	0	-	No
		7.9 (UL)	5	0	-	No
			15	0	-	No
			35	0	-	No
	40	6.7 (LL)	5	0	-	No
			15	0	-	No
			35	0	-	No
		7.9 (UL)	5	0	-	No
			15	0	-	No
			35	0	-	No
MCE	10	7.1 (LL)	5	7	0.89	Yes
			15	4	1.01	Yes
			35	1	0.91	Yes
		7.9 (UL)	5	10	0.92	Yes
			15	7	0.95	Yes
			35	4	0.95	Yes
	40	7.1 (LL)	5	0	-	No
			15	0	-	No
			35	0	-	No
		7.9 (UL)	5	0	-	No
			15	0	-	No
			35	0	-	No

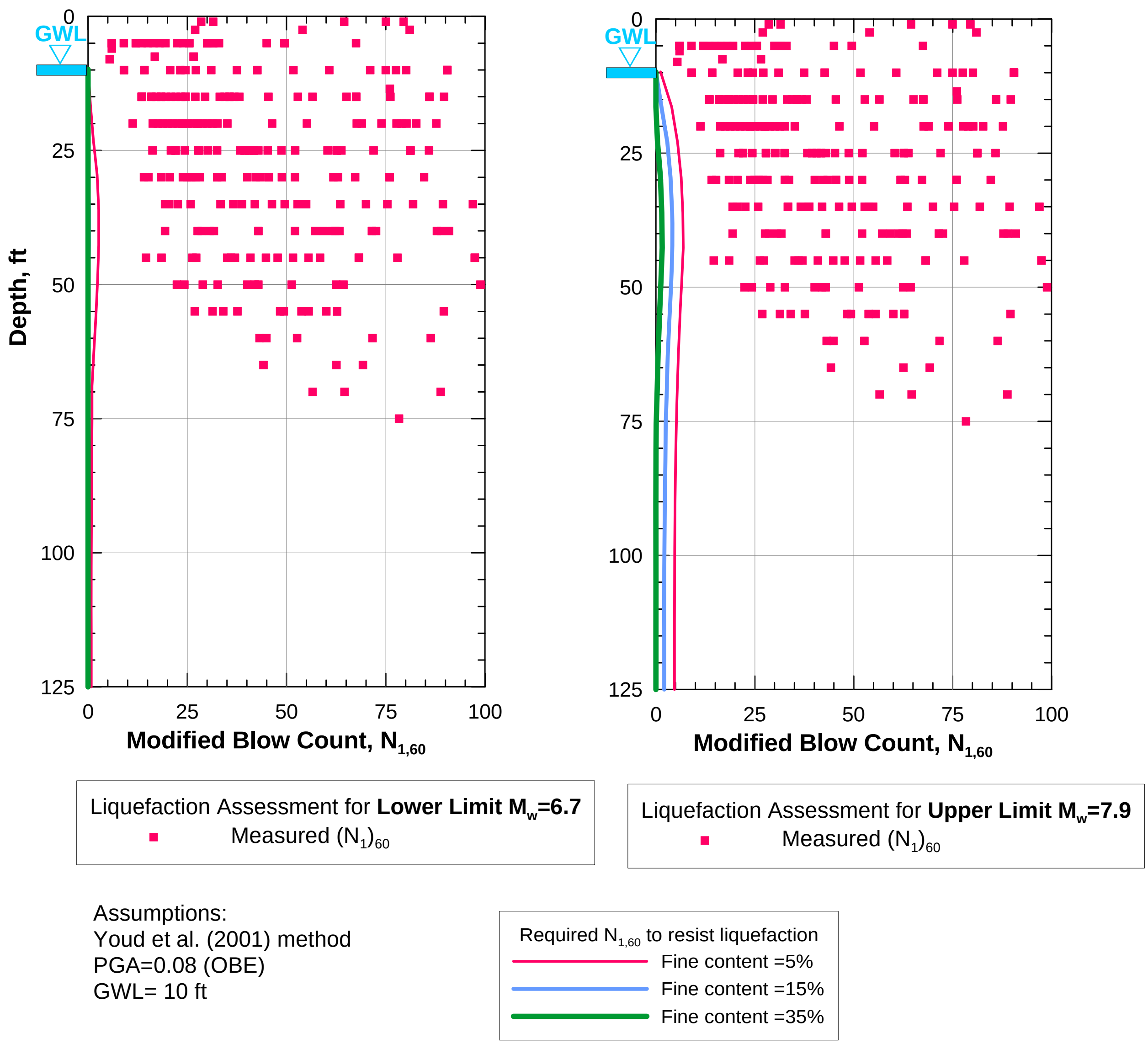


Figure 6.3-1
Liquefaction Assessment Results (Youd et al. [2001], OBE, GWL = 10 ft)

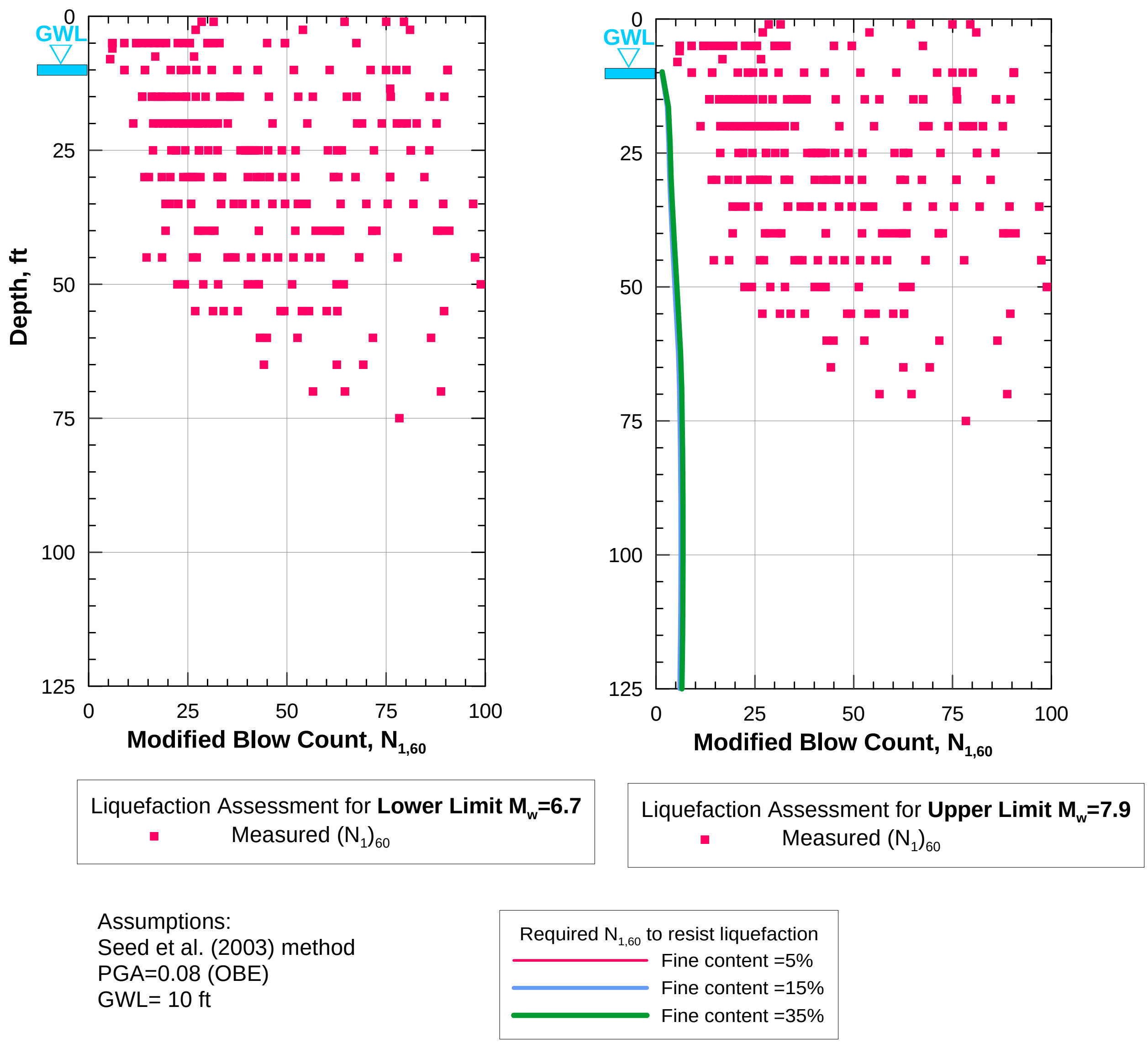
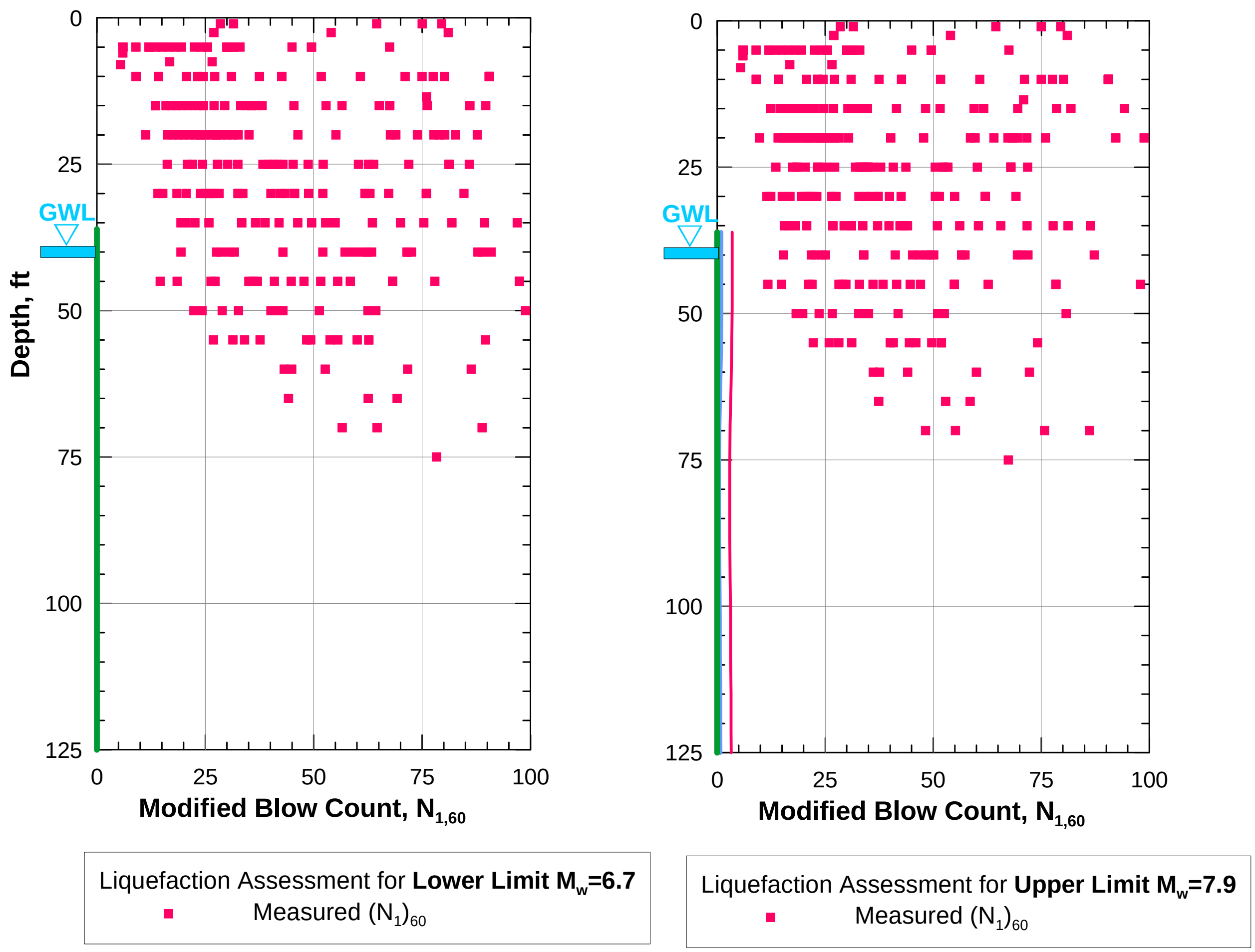


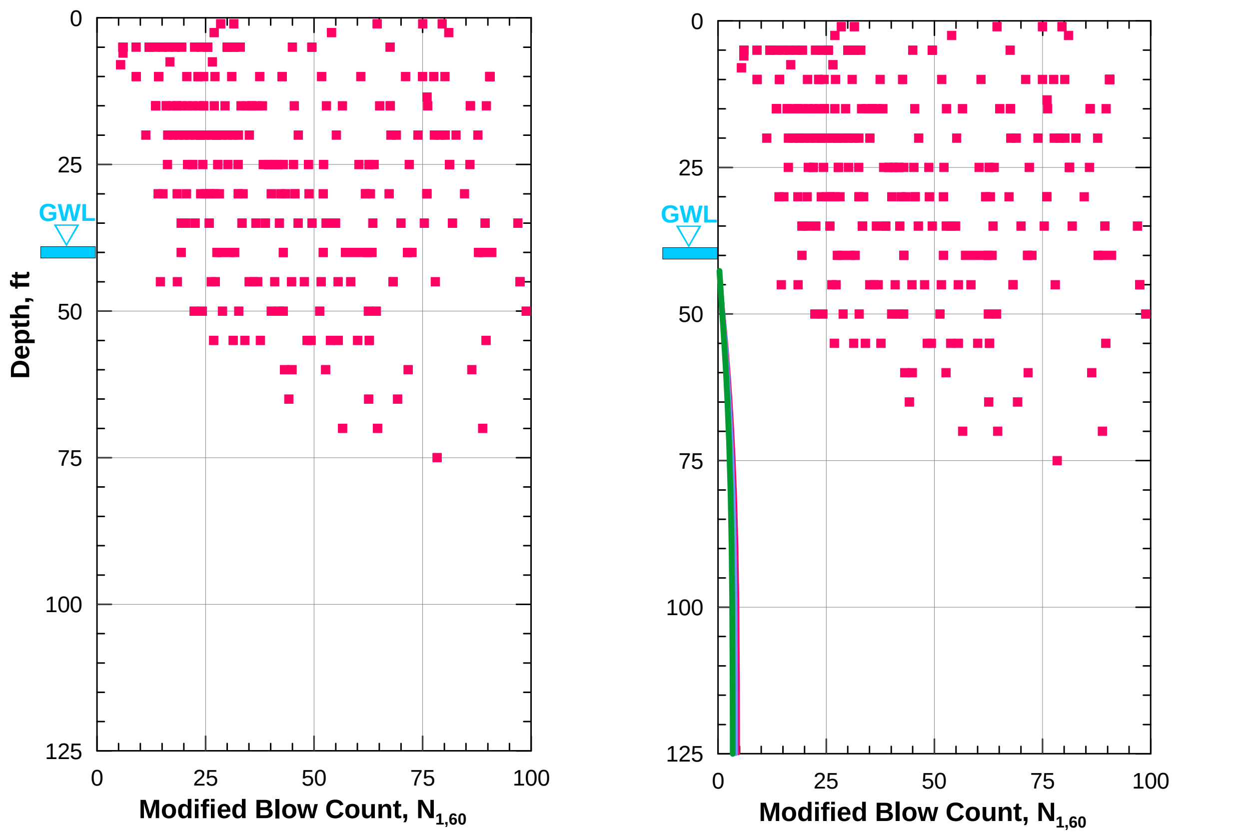
Figure 6.3-2
Liquefaction Assessment Results (Seed et al. [2003], OBE, GWL = 10 ft)



Assumptions:
Youd et al. (2001) method
PGA=0.08(OBE)
GWL= 40 ft

Required $N_{1,60}$ to resist liquefaction
Fine content =5%
Fine content =15%
Fine content =35%

Figure 6.3-3
Liquefaction Assessment Results (Youd et al. [2001], OBE, GWL = 40 ft)



Liquefaction Assessment for **Lower Limit M_w=6.7**
Measured $(N_1)_{60}$

Liquefaction Assessment for **Upper Limit M_w=7.9**
Measured $(N_1)_{60}$

Assumptions:
Seed et al. (2003) method
PGA=0.08 (OBE)
GWL= 40 ft

Required $N_{1,60}$ to resist liquefaction
Fine content =5%
Fine content =15%
Fine content =35%

Figure 6.3-4
Liquefaction Assessment Results (Seed et al. [2003], OBE, GWL = 40 ft)

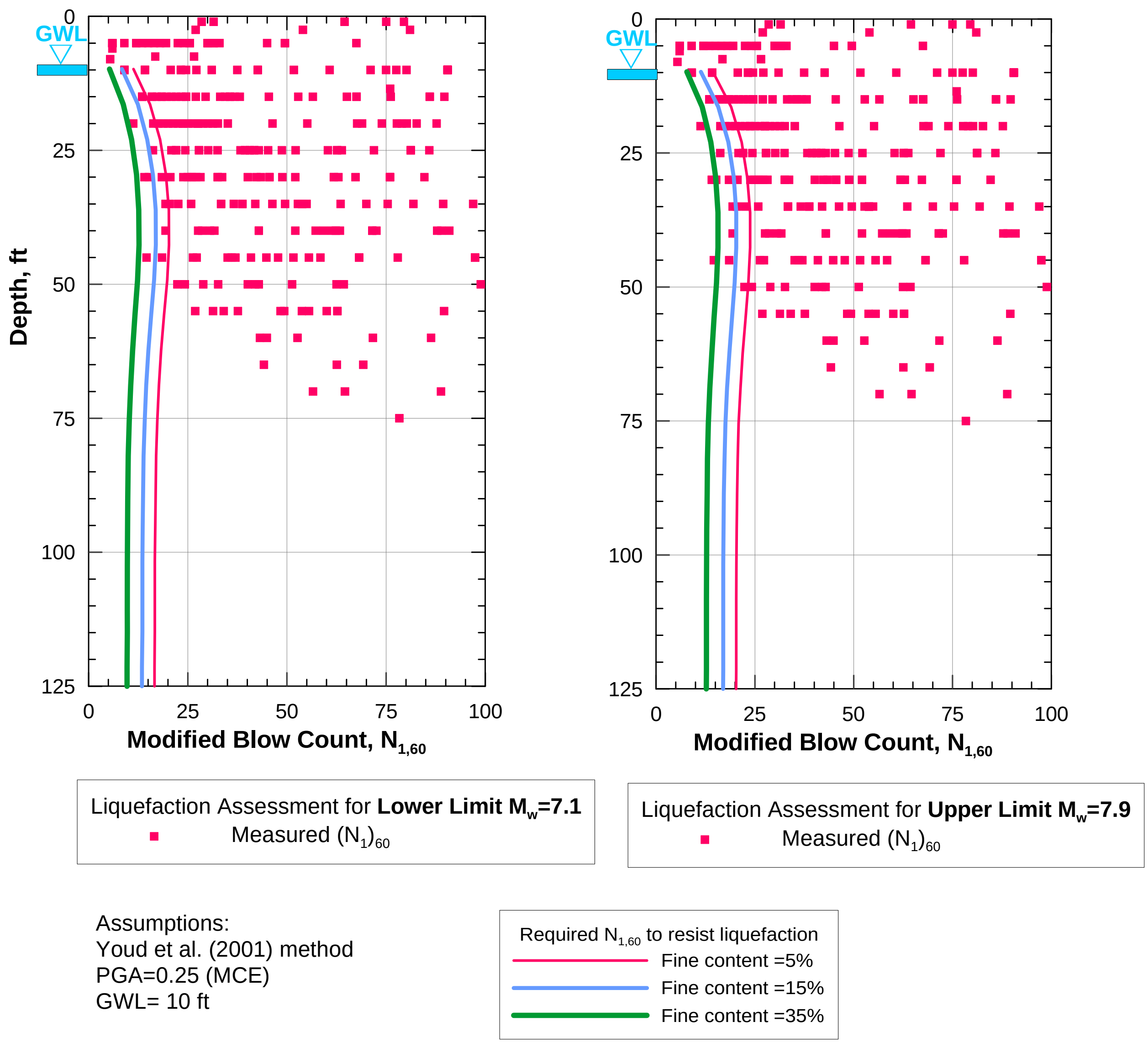


Figure 6.3-5
Liquefaction Assessment Results (Youd et al. [2001], MCE, GWL = 10 ft)

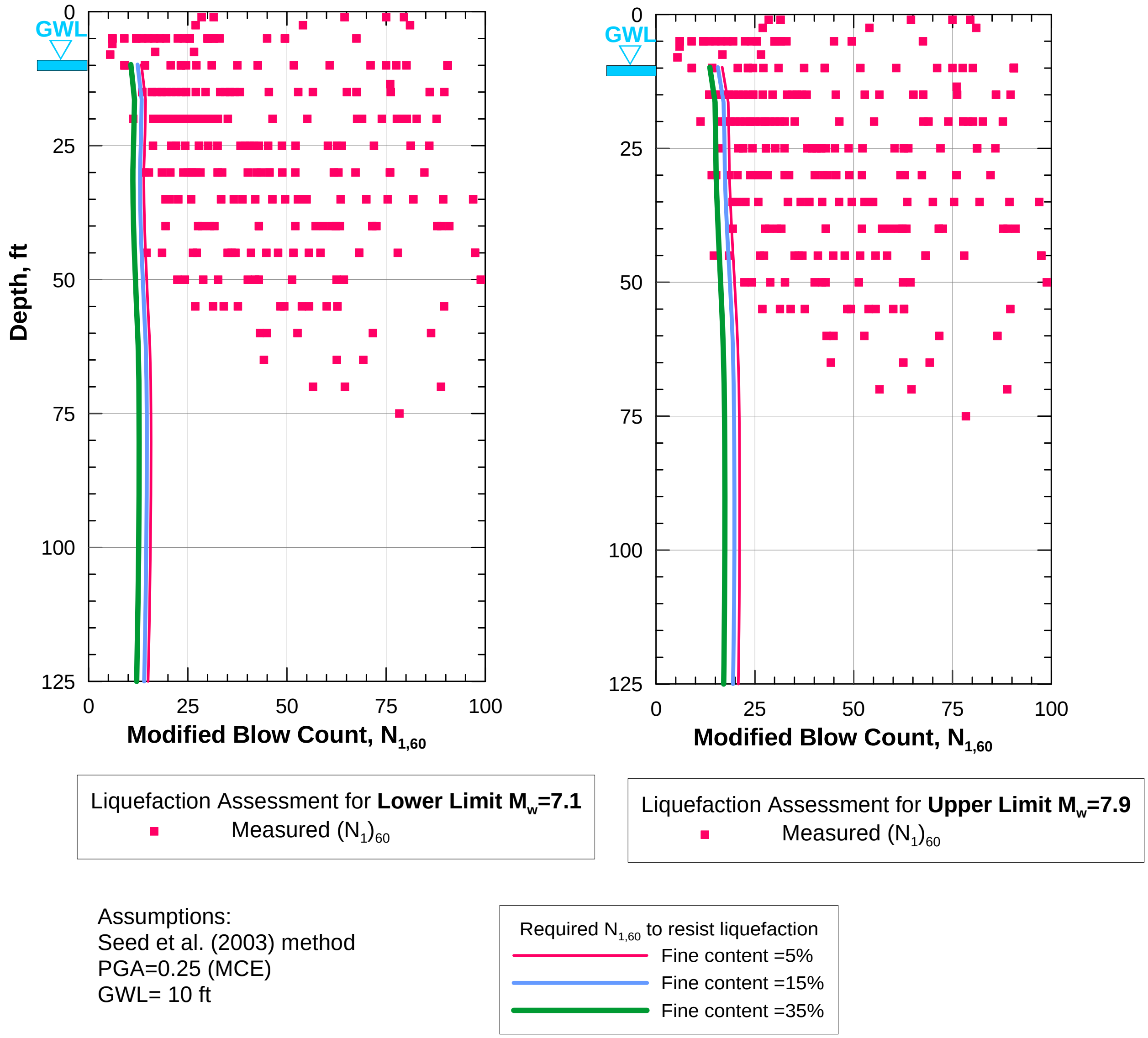


Figure 6.3-6
Liquefaction Assessment Results (Seed et al. [2003], MCE, GWL = 10 ft)

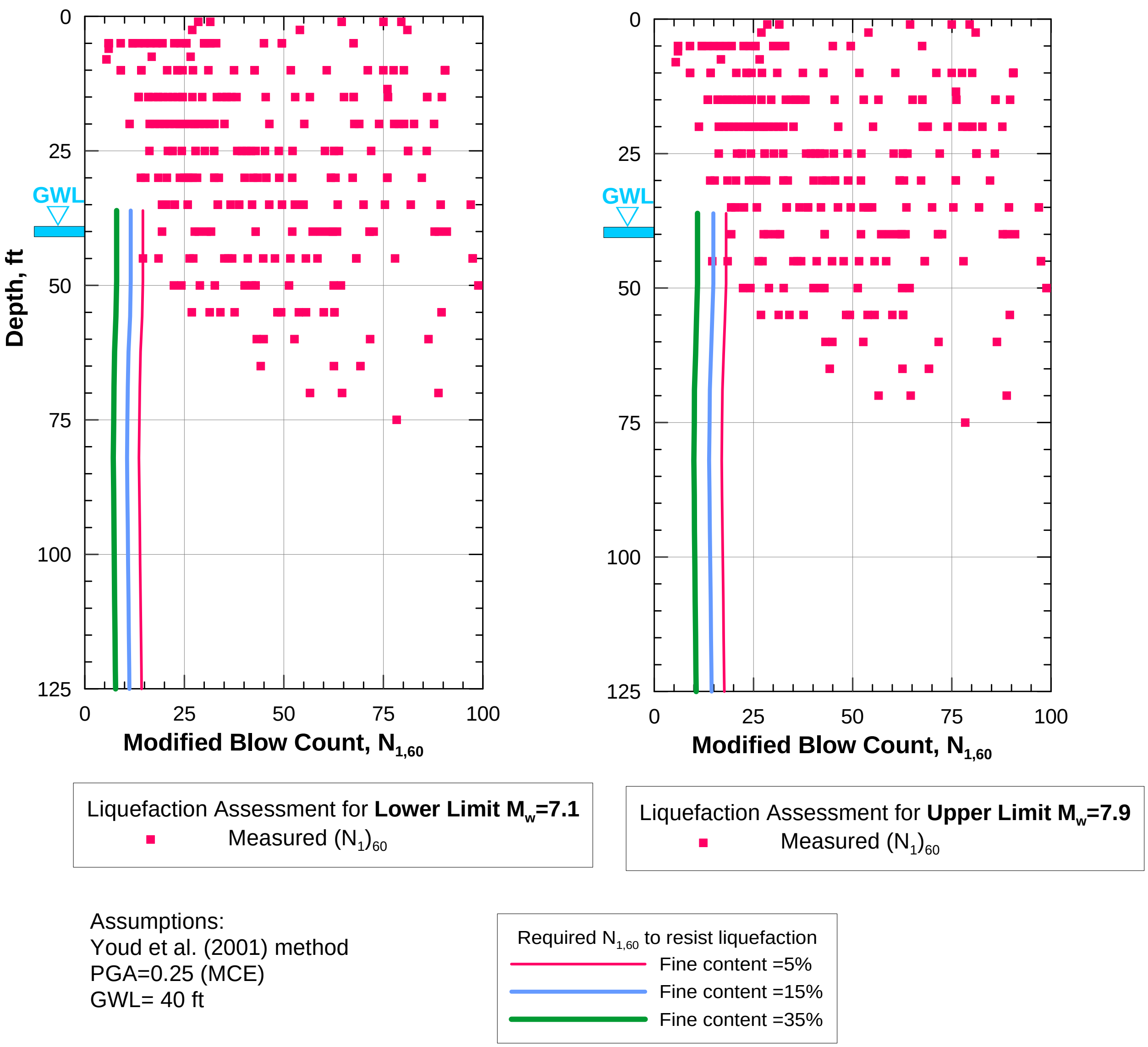


Figure 6.3-7
Liquefaction Assessment Results (Youd et al. [2001], MCE, GWL = 40 ft)

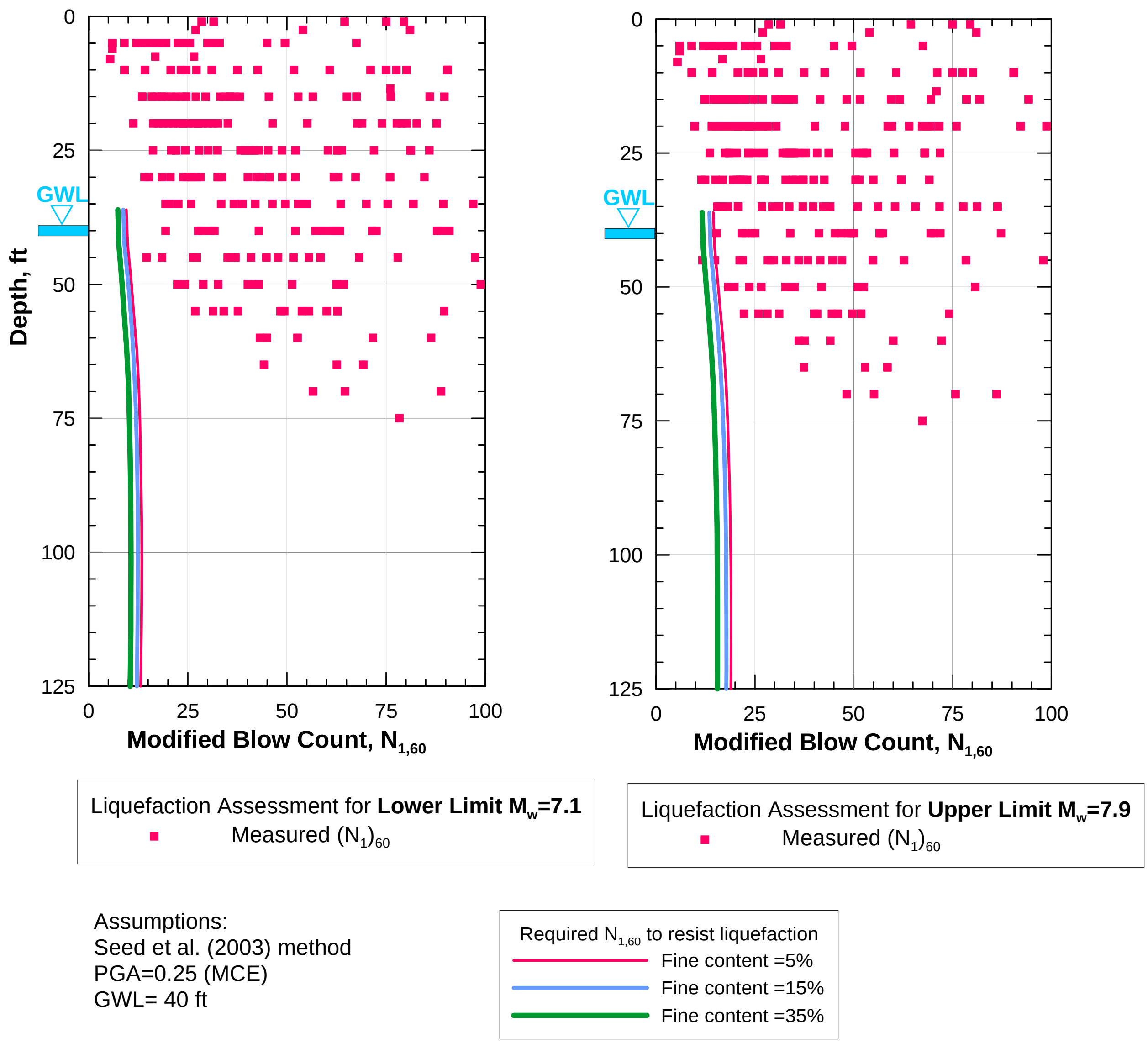


Figure 6.3-8
Liquefaction Assessment Results (Seed et al. [2003], MCE, GWL = 40 ft)

6.4 Conclusions

Using the available SPT data from the historical borehole database at the vicinity of the Fresno section (Package 1) of the alignment, the PMT assessed the liquefaction based on the simplified methods of Youd et al. (2001) and Seed et al. (2003). A summary of the average FOS resulting from liquefaction assessment is presented in Table 6.3-1.

This data suggests the moderate liquefaction potential for the case of the MCE with an upper-limit magnitude of 7.9 and groundwater at a depth of 10 feet. As shown, the liquefaction susceptibility decreases with increasing fines content.

Liquefaction is not considered a hazard for both lower- and upper-limit magnitudes of the OBE with groundwater at a depth of 10 and 40 feet or for both magnitudes of the MCE with groundwater depth at 40 feet.

6.5 Seismic Deformations

TM 2.9.10 requires consideration of seismic deformations for only the OBE event. Based on the liquefaction assessment above, the study area is not considered particularly prone to seismically induced deformations, including liquefaction of unsaturated (dry) soils above the level of seasonal groundwater fluctuation.

7.0 Design

7.1 Fresno Street Overpass

Based on the load demand and compatibility with the foundation for standard structures as presented by the Engineering Management Team (EMT), it is proposed that the complex and nonstandard bridges within the study area be supported on drilled (cast-in-drilled-hole [CIDH]) shafts. The designs of these drilled shafts are based on the design soil profile presented above and the following TMs:

- TM 2.9.10 – Geotechnical Analysis and Design Guidelines
- TM 2.3.2 – Structure Design Loads

The HST project makes use of the Load and Resistance Factor Design (LRFD) methodology for the engineering design approach in both geotechnical analysis and structural engineering. The following summarizes the 30% Design requirements and analyses required under LRFD.

7.1.1 LRFD Methodology

Section 6.3.3 of TM 2.9.10 and TM 2.3.2 – Structural Loads require that for structures carrying HSTs, the design shall be in accordance with California Bridge Design Specifications: AASHTO LRFD Bridge Design Specifications, with California Amendments (CBDS).

Per section 6.7 of TM 2.3.2, the governing formula for LRFD design is as follows:

$$\Sigma\, \eta_i\, \gamma_i\, Q_i \leq \Phi R_n = R_r$$

Where

γ_i = load factor applied to force effects

Φ = resistance factor applied to minimal resistance

η_i = load modifier relating to ductility, redundancy, and importance

Q_i = force effect or service load

R_n = nominal resistance

R_r = factored resistance, ΦR_n

For loads for which a maximum value of η_i produces an unfavorable action, the value of η_i shall be equal to 1.05 to account for the 100-year design life of the facility.

7.1.2 Limit States

TM 2.3.2 -Structure Design Loads mandates that at least four limit states be considered for the 30% Design stage:

1. The Service Limit States. The Service Limit State evaluates foundation deformation at the operational use level such as the foundation settlement and the horizontal movement under static loading. Loads under this limit state are termed "service loads" and are equivalent to "design loads" used under the Allowable Strength Design (ASD) methodologies.

2. The Buoyancy Limit State. The Buoyancy Limit State evaluation is for uplift with a minimum weight structure in the case of dewatering shut off. The subterranean structures within this section include the Fresno Grade Separation.

3. The Strength Limit States. The Strength Limit evaluates the bearing capacity of the foundation when subjected to static loading condition and, for the Strength 5 Limit State, operating basis seismic events (OBE).

4. The Extreme Event Limit States. Extreme Limit States evaluate seismic and derailment loading conditions. Extreme 3 is based on the MCE which is the design earthquake for the NCL performance level.

7.1.3 Displacement Criteria

Limits for tolerable foundation settlements and displacements are presented in TM 2.9.10 – Geotechnical Design Guidelines and supersede criteria indicated in CBDS and the California Amendments. For deep foundations, tolerable settlements or displacements are measured at the top of the foundation: the pile cap, pile head, or the ground surface for CIDH extensions. Limiting values for allowable deformations that are based on tolerable movements for the proposed HST bridges and tracks are in development. Tolerable settlement or displacement criteria are prescribed in Table 6.3.5-1 of TM 2.9.10.

7.1.4 CIDH Resistance Factors

As per Section 6.3.3 of TM 2.9.10, the CBDS resistance factors for CIDH piles at the Strength and Extreme limit states are shown in Table 7.1-1. No resistance factors are applied to the Service 1 limit state since this limit state is reserved for evaluation of displacement criteria under service loads.

Table 7.1-1
Resistance Factor for Single CIDH Piles

	Shaft	Tip	Uplift
Strength	0.7	0.5	0.7
Extreme	1.0	1.0	1.0

In accordance with CBDS, the resistance factor used in lateral load analyses of CIDH piles is unity.

7.1.5 CIDH Nominal Axial Resistance

The analysis for HST structures relies on skin friction and end bearing. The governing relationship for determining the nominal axial resistance is expressed in accordance with AASHTO 2010 (Section 10.8.3.5) as follows:

$$R_R = \Phi_{qp} R_p + \Phi_{qs} R_s$$

In which

$R_p = q_p A_p$
$R_s = q_s A_s$

Where

R_p = nominal shaft tip resistance (kips)
R_s = nominal shaft side resistance (kips)
Φ = resistance factor (Table 7.1.4-1)
q_p = unit tip resistance (kips)
q_s = unit side resistance (kips)
A_p = area of shaft tip (ft^2)
A_s = area of shaft side surface (ft^2)

7.1.5.1 Cohesionless Soils

The nominal side resistance in cohesionless soil is estimated following the O'Neill and Reese (1999) formulation, specified in the AASHTO 2010 (Section 10.8.3.5.2b) as follows:

$$q_s = \beta\sigma'_v \leq 4.0 \quad \text{for } 0.25 \leq \beta \leq 1.2$$

In which

$\beta = 1.5 - 0.135\sqrt{z}$ for $N_{60} \geq 15$

$\beta = \frac{N_{60}}{15}(1.5 - 0.135\sqrt{z})$ for $N_{60} < 15$

Where

σ'_v = vertical effective stress at soil layer mid-depth (ksf)
β = load transfer coefficient
z = depth below ground, at soil layer mid-depth (ft)
N_{60} = average SPT blow count (corrected only for hammer efficiency) in the design zone under consideration (blows/ft)

The nominal tip resistance in cohesionless soil is estimated following the O'Neill and Reese (1999) formulation as specified in the 2010 AASHTO (Section 10.8.3.5.2c) as follows:

$$q_p = 1.2N_{60} \quad \text{for } N_{60} \leq 50$$

Where

N_{60} = average SPT blow count (corrected only for hammer efficiency) in the design zone under consideration (blows/ft)

7.1.5.2 Uplift Resistance

The uplift resistance of the pile is estimated following the O'Neill and Reese (1999) formulation as specified in the 2010 AASHTO (Section 10.8.3.7) as follows:

$$R_R = \varphi_{up} R_n$$

Where

R_R = Factored uplift resistance (kips)
R_n = Nominal uplift resistance due to side resistance (kips)
φ_{up} = Resistance factor for uplift resistance

After estimating the uplift resistance of the pile or pile group, the buoyant weight is added as an additional resisting force.

7.1.5.3 Axial Group Reduction Factors

The efficiency of a pile group under axial loading conditions is determined by the on-center spacing between the piles. AASHTO 2010 (Section 10.8.3.6.3 – Cohesionless Soil) recommends a group reduction factor of 0.65 for on-center spacings of 2.5 pile diameters and 1.0 for on-center spacings of four pile diameters or more. Group reduction factors between these spacings are determined by linear

interpolation. For 30% Design, the piles have been spaced at a minimum center-to-center spacing of four pile diameters and no axial group reduction factor applied.

7.1.5.4 Geotechnical Axial Resistance of Piles

Using the ground models and the LRFD procedures presented above, the factored nominal axial resistance for the typical foundation types are calculated using a spreadsheet. Table 7.1-2 shows the factored resistance of 3.0 feet diameter CIDH piles for various locations and limit states.

Table 7.1-2
CIDH Factored Axial Resistances

Location	Limit State	Pile Length (ft)	Factored Compression Resistance (kips)	Factored Tension Resistance (kips)
Fresno Street	Strength	145	2050	1950
	Extreme	145	7750	6200

7.1.6 Lateral Load Analyses

Lateral pile analyses were performed using the computer programs LPILE6 and GROUP8 for single piles and group of piles, respectively. LPILE6 was used to calculate soil stiffness along the length of the pile. The soil stiffness was then provided as springs to be modeled in the structural model. A demand analysis was performed using GROUP8 to evaluate pile group interaction effects and to obtain axial, bending moment, shear, and displacement demands on the piles based on various limit states. Soil parameters used in the lateral load analyses are summarized in Table 7.1-3.

Table 7.1-3
Soil Parameters for Lateral Resistance Design

Soil Design Parameters	Layer A	Layer B	Layer C
Soil Model	API Sand	API Sand	API Sand
Thickness of layer (ft)	35	25	>70
Friction Angle, ϕ' (deg)	37	35	37
Total Unit Weight, γ (pcf)	125	125	125
Soil Modulus Parameter, k (pci)	225	90	125

The GROUP8 results for the Fresno Street Overpass are summarized in Tables 7.1-4 and 7.1-5. Based on the results above, the maximum relative horizontal displacement between the bottom and top of pile for Strength 5 limit state (OBE loading) is less than 1.75 inches, per Section 6.3.4 of TM 2.9.10.

Table 7.1-4
Pile Demand for Various Limit States

Structure	Foundation	Compression Load Demand (kips)		Tension Load Demand (kips)	
		Strength 1 & 5	Extreme	Strength 1 & 5	Extreme
Fresno Street	4-leading row, 3-trailing row – 3.0-ft diameter CIDH	2025	1900	1015	1445

Table 7.1-5
Lateral Deflection for Various Limit States

Structure	Foundation	Lateral Deflection (in)
		Strength 5
Fresno Street	4-leading row, 3-trailing row – 3.0 ft diameter CIDH	0.3

7.1.7 Lateral Group Reduction Factors

The efficiency of a pile group under lateral loading conditions is determined by the on-center spacing between the piles. AASHTO 2010 (Section 10.7.1.4 – Horizontal Pile Foundation movement) recommends the following for pile P-multipliers:

Table 7.1-6
Pile P-Multipliers for Multiple Row Shading

Pile Center-to-Center Spacing (in the direction of loading)	P-Multiplier		
	Row 1	Row 2	Row 3
3 Diameters	0.8	0.4	0.3
5 Diameters	1.0	0.85	0.7

Refer to Figure 10.7.2.4-1- Definition of Loading Direction and Spacing for Group Effects for details of applied load direction and pile configuration.

7.1.8 Vertical Displacements

The vertical displacement of the foundation is evaluated based on the Service 1 limit state loading condition and includes both elastic compression of the pile as well deformation of the soil through interface stiffness and end bearing compression, i.e., settlement. The total vertical displacement is estimated by adding the elastic settlement in the pile to the settlement under service loads. As the drilled shafts are not anticipated to be underlain by compressible (cohesive) soil, the consolidation settlement is not estimated. The settlement calculated using GROUP8 is summarized in Table 7.1-7.

Table 7.1-7
Estimated Total Settlement at Pile Cap Level

Structure	Vertical Displacement (in)
Fresno Street HST Overpass	< 0.50

7.2 Fresno Grade Separation

The predominantly granular nature of the ground conditions at the Fresno Grade Separation location favor continuous wall types without gaps to mitigate possible ground loss. Given the proximity of the HST alignment to adjacent structures, the Union Pacific Railroad, and potential shallow groundwater conditions due to Dry Creek Canal and groundwater detention basins, "stiff" excavation support systems capable of minimizing water inflows are anticipated.

The most suitable wall components are therefore expected to be either diaphragm slurry wall, secant pile, or a deep soil mixing/cast-in-place (temporary/permanent) wall combination. A soldier-pile-and-lagging/cast-in-place wall combination is also a possibility at dry locations or where adequate dewatering is employed. For 30% Design, a secant pile wall with a diameter of 3 feet and center-to-center spacing of 2.5 feet was determined to be appropriate as a temporary excavation support system (ESS) during construction.

Loads on the temporary shoring wall were developed in accordance with Section 6.7 of TM 2.9.10. For this purpose, excavation support modeling was performed using the computer program Oasys FREW, which models the excavation construction sequence, incorporating the following:

- Effects of surcharge loading
- Construction dewatering
- Temporary and permanent struts
- Excavation support wall stiffness
- Different soil layers and properties

The ESS consists of a temporary secant pile wall and the base slab. More specifically, the stiffness of the secant pile wall was estimated assuming that all the piles are composed of reinforced concrete and the area of steel is 1% of the total area of the pile cross section. The combined Young's modulus of the composite cross section of each pile was then calculated based on the above assumption, and finally, the stiffness of the secant pile wall was estimated based on a center-to-center spacing of 2.5 feet for the piles. The base slab stiffness was estimated assuming a slab thickness of 4 feet. The secant pile wall and base slab stiffness calculations used in the software were evaluated using concrete Young's modulus for long-term conditions.

Temporary strut stiffness was estimated assuming 24-inch steel pipe struts spaced 15 feet apart horizontally. The size of the pipe struts was estimated through an iterative process, to verify that the struts can sustain the anticipated applied axial loads without buckling. The permanent struts, used throughout the Fresno Grade Separation except for the jacked box location, were modeled as concrete beams, having a 4 feet by 4 feet cross section and 15 feet spacing along the excavation. The individual permanent strut stiffness was estimated based on this configuration using concrete Young's modulus for long-term conditions.

Groundwater conditions inside and outside of the excavation support system were also modeled. Groundwater table outside the excavation was modeled at a constant depth of 10 feet below existing ground surface. During modeling of the excavation, groundwater was maintained at 2 feet below the

bottom of the excavation for each excavation stage. Because of the granular nature of soils at the Grade Separation location, all the analyses were performed for drained conditions.

A surcharge was included in the models to simulate embankment and construction equipment. The surcharge was 30 feet wide at 15 feet from the edge of the excavation.

7.2.1 Lateral Earth Pressures

According to the SC Solutions (2011) seismic modeling, the study area is in a seismic zone with PGAs 0.08g and 0.28g, respectively. Therefore, per Section 6.10.13 of TM 2.9.10, seismic earth pressures are not considered for either rigid or yielding walls for the permanent wall design. Table 7.2-1 shows the lateral earth pressure coefficients that were used for the permanent wall design of the Fresno Grade Separation structure.

Table 7.2-1
Earth Pressures

Type of Wall	Lateral Earth Pressure Coefficients
Unrestrained Permanent Trench Wall	Active condition, $k_a = \tan^2(45 - \phi'/2)$
Restrained Permanent Trench Wall	At-rest condition $k_o = 1 - \sin \phi'$

7.2.2 Results of Excavation Sequence Modeling

A total of four sections were modeled in FREW for the Fresno Grade Separation structure based on the two design soil profiles presented in Section 4.5. Table 7.2-2 presents the stations where each section is applicable. The ground conditions change at Station 10924+00; thus, Sections 1 and 2 located north of this station have the same soil parameters. Sections 3 and 4, extending south of Station 10924+00, refer to the same ground conditions. However, Section 4 was specifically developed for the case of the jacked box. Tables 7.2-3 and 7.2-4 summarize the soil parameters used for each model.

Table 7.2-2
Section Stationing

Section	From Station	To Station
1	10885+00	10913+00
2	10913+00	10924+00
3	10924+00	10975+00
4	10935+95	10939+40

Table 7.2-3
Grade Separation FREW Input for Sections 1 and 2

Layer	Thickness (ft)	γ (pcf)	φ' (deg)	E (ksf)
Silty Sand (Layer A)	40	125	35	800
Silty Sand (Layer B)	>60	125	40	1540

Table 7.2-4
Grade Separation FREW Input for Sections 3 and 4

Layer	Thickness (ft)	γ (pcf)	φ' (deg)	E (ksf)
Silty Sand (Layer A)	10	125	37	940
Silty Sand (Layer B)	10	125	40	1540
Silty Sand (Layer C)	>80	125	39	1420

Tables 7.2-5, 7.2-6, and 7.2-7 summarize the construction sequence modeled in FREW for all sections.

Table 7.2-5
Section 1 Construction Sequence as Modeled in FREW

Stages	Section 1 (EL) **Sta. 10885+00 – 10913+00**
1. Install secant pile wall	220
2. Excavate Dewater	281 279
3. Install Strut 1 and preload 50%	283
4. Excavate Dewater	265 263
5. Install Strut 2 and preload 50%	267
6. Excavate to Bottom of excavation Dewater	248 246
7. Install base slab	251
8. Partially construct U-wall	From 253.437 to 265
9. Remove Strut 2	267
10. Complete Permanent Wall	From 265 to 278
11. Install Permanent Strut 3	280
12. Remove Strut 1	283

Table 7.2-6
Section 2 and 3 Construction Sequence as Modeled in FREW

Stages	Section 2 (EL) Sta. 10913+00 – 10924+00	Section 3 (EL) Sta. 10924+00 – 10975+00
1. Install secant pile wall	210	210
2. Excavate	284	284
3. Install Strut 1 and preload 50%	286	286
4. Excavate Dewater	268 266	268 266
5. Install Strut 2 and preload 50%	270	270
6. Excavate Dewater	253 251	253 251
7. Install Strut 3 and preload 50%	255	255
8. Excavate to Bottom of excavation Dewater	239.5 237.5	238 236
9. Install base slab	242.6	241
10. Partially construct permanent wall	From 245.133 to 253	From 243.508 to 253

Stages	Section 2 (EL) Sta. 10913+00 – 10924+00	Section 3 (EL) Sta. 10924+00 – 10975+00
11. Remove Strut 3	255	255
12. Partially construct permanent wall	From 253 to 268	From 253 to 268
13. Remove Strut 2	270	270
14. Complete permanent wall	From 268 to 281	From 268 to 281
15. Install Permanent Strut	283	283
16. Remove Strut 1	286	286

Table 7.2-7
Section 4 Construction Sequence as Modeled in FREW

Stages	Section 4 (EL) Sta. 10935+95 – 10939+40
1. Install secant pile wall	299 to 220
2. Excavate both sides of box	289 (ground level)
3. Excavate	282
4. Install Strut 1 and preload 50%	284
5. Excavate Dewater	267 265
6. Install Strut 2 and preload 50%	269
7. Excavate Dewater	256 254
8. Install Strut 3 and preload 50%	258
9. Excavate to Bottom of excavation Dewater	244 242
10. Install base slab	246.5
11. Remove Strut 3	258
12. Remove Strut 2	269
13. Install Strut 4 and preload 10%	298
14. Remove Strut 1	284

The results of the analyses are summarized in Table 7.2-8 and show that deflections and bending moments are within the structural limits of the system. The allowable bending moments for the ESS are presented in Table 7.2-8.

Table 7.2-8
Excavation Sequencing Modeling Results

Section	Embedment Depth of ESS Wall Below Bottom of Excavation (ft)	Maximum Deflection of ESS Wall (in)	Service Bending Moment of ESS Wall (kip-ft/ft)
1	28	0.6	125
2	30	1.2	235
3	28	1.3	250
4	24	1.5	235

7.2.3 Base Stability Evaluation

The base stability of the temporary excavation is evaluated to avoid instability due to seepage from construction dewatering. The excavation is considered safe against base instability for an FOS greater than 2. The FOS is defined as the ratio of the critical hydraulic gradient over the hydraulic gradient of flow exiting the base:

$$FOS = \frac{i_c}{i_e} = \frac{\frac{\gamma_T}{\gamma_w} - 1}{\frac{\Delta h}{\Delta L}} > 2$$

Where

i_c = critical hydraulic gradient
i_e = exit hydraulic gradient
γ_T = total unit weight of soil
γ_w = total unit weight of water
Δh = difference in hydraulic head over distance ΔL

The numerator of the above equation is approximately 1, since the total unit weight of soil γ_T is about twice the unit weight of water γ_w. From the above equation for a factor of safety of 2, it can be shown that the minimum wall embedment to ensure base stability is half the difference in groundwater level of the two sides of the wall:

$$h_e \geq \frac{H_w}{2}$$

Where

h_e = wall embedment below base of excavation
H_w = groundwater level difference between inside and outside of the U-Walls

Table 7.2-9 presents the wall embedment as well as the groundwater level differences for the four sections. It can be observed that for all four cases the wall embedment is adequate and base instability is avoided.

Table 7.2-9
Base Stability Evaluation for the Four Sections

Section	Embedment depth, h_e (ft)	$H_w/2$
1	28	17
2	30	21
3	28	22
4	24	18.5

7.2.4 Tie-Downs/Tension Piles

Over certain sections of the Fresno Grade Separation, there is a possibility that groundwater may be encountered at shallower depths of around 10 feet below existing ground surface. Should this occur, tie-down or tension piles will be required to counteract the water buoyancy forces. The design soil profile for the area of high groundwater is as provided Table 5.5-2. Design of tie-downs is based on the concept of Section 10.9 of AASHTO – Micropiles.

For this design, Type A micropile in very dense sand was used, which yields an ultimate bond stress of 3 kips per square foot. Applying a resistance factor of 0.7 per California Amendments to AASHTO yields a factored bond stress of 2.1 kips per square foot.

To resist a hydrostatic pressure equivalent to 30 feet of water, it is recommended that two rows of tie-downs, one below each track, spaced at 5 feet center-to-center with a minimum embedment length of 35 feet be provided.

8.0 Limitations and Further Information

The 30% Design effort is based on limited information included in historical geotechnical reports. In many cases the information used is from depths that are much shallower than the anticipated CIDH pile lengths and depth of Fresno Grade Separation excavation. In addition, the information is spatially removed from the alignment and there is substantial evidence that the soils vary considerably in the both horizontal and vertical directions. The results of this memorandum should be considered preliminary and refined by the Contractor during final design once site-specific information gathered by the Contractor is available. Site-specific information will be based on 30% geotechnical investigation conducted by the Regional Consultant and will be provided to the Contractor in the form of Geotechnical Data Report and Geotechnical Baseline Report.

9.0 References

American Association of State Highway and Transportation Officials, 2010. LRFD Bridge Design Specifications. 5th Edition.

Caltrans, 1953a. Clinton Avenue Overcrossing, Log of Test Borings.

_______, 1953b. North Fresno Undercrossing, Log of Test Borings.

_______, 1963, South Fresno Viaduct, Log of Test Borings.

_______, 1990. Clinton Ave. O.C. (Widen), Log of Test Borings.

_______1996. Caltrans Seismic Hazards Map and A Technical Report to Accompany the Caltrans California Seismic Hazards Map (Based on Maximum Credible Earthquakes).

California Geological Survey, 1997 (rev.2008). Guidelines for Evaluating and Mitigating Seismic Hazards in California, CDMG Special Publication 117.

_______, 2009. Alquist-Priolo Earthquake Fault Zones. Electronic document, available at: http://www.conservation.ca.gov/cgs/rghm/ap/Pages/Index.aspx

_______, 2010. Fault Activity Map of California. Geologic Data Map No. 6. Electronic document, available at: http://www.quake.ca.gov/gmaps/FAM/faultactivitymap.html

Central Valley Testing, Inc., 2005. Geotechnical Engineering Investigation, Tract No. 5614, West Bullard Avenue West of North Grantland Avenue, Fresno, California. Prepared for Sterling Development.

City of Fresno, 2010. Water Quality Annual Report 2010.

Fresno Irrigation District (FID) et. al., 2006. Fresno Area Regional Groundwater Management Plan.

Huntington, Gordon L., 1971. Soil Survey, Eastern Fresno Area, California. United State Department of Agriculture, Soil Conservation Service.

Jenkins, O.P., 1965. Geological Map of California Fresno Sheet, Division of Mines and Geology.

Jennings, C.W., 1994. Fault activity map of California and adjacent areas with locations and ages of recent volcanic eruptions: California Department of Conservation, Division of Mines and Geology Data Map Series No. 6, 92 p., 2 plates, map scale 1:750,000

Krazan & Associates, Inc., 1987. Preliminary Geotechnical Engineering Investigation, Tract 3881, Sommerset No. 3. Fresno, California. Prepared for Keterson Development.

_______, 1991a. Preliminary Geotechnical Engineering Investigation, Tract No. 4229, Fresno, California. Prepared for Trend Homes.

_______, 1991b. Preliminary Geotechnical Engineering Investigation, Tract No.4304, Fresno, California. Prepared for Trend Homes.

_______, 1991c. Preliminary Geotechnical Engineering Investigation, Tract No. 4320, Fresno, California. Prepared for Trend Homes.

_______, 1993. Preliminary Geotechnical Engineering Investigation, Proposed Residential Development Tract 5414, Cornelia Avenue Near McKinley Avenue, Fresno, California. Prepared for DS Ventures, LLC.

_______, 1994a. Preliminary Geotechnical Engineering Investigation, Tract No.4384, Fresno, California. Prepared for Trend Homes.

_______, 1994b. Preliminary Geotechnical Engineering Investigation, Proposed Residential Development Tract 5325, West Gettysburg and North Polk Avenues, Fresno, California. Prepared for Century Builders.

_______, 1997. Preliminary Geotechnical Engineering Investigation, Proposed Residential Development Tentative Tract No. 5512, Cornelia Avenue Near Clinton Avenue, Fresno, California.Prepared for Rabe Engineering, Inc.

Page, R.W., 1986. Geology of the fresh ground-water basin of the Central Valley, California, with texture maps and sections: United States Geological Survey, Professional Paper 1401-C.

O'Neill, M.W., and Reese, L.C., 1999. Drilled Shafts: Construction Procedures and Design Methods. FHWA-IF-99-025.

SC Solutions, 2011. California High-Speed Train Project – 30% Design Ground Motions.

Seed, R.B., Cetin, K.O., Moss, R.E.S., Kammerer, A.M., Wu, J., Pestana, J.M., Riemer, M.F., Sancio, R.B., Bray, J.D, Kayen, R.E, and Faris, A., 2003. Recent Advances in Soil Liquefaction Engineering: A unified and consistent framework. 26th Annual ASCE Los Angeles Geotechnical Spring Seminar, Keynote Presentation, H.M.S Queen Mary, Long Beach, California.

Soils Engineering, Inc., 2005. Preliminary Soils Investigation for Vesting Tentative Tract No. 5316 Located at the Southeast Corner of West Dakota Avenue and North Hayes Avenue in Fresno, CA. Prepared for Centex Homes.

Technicon Engineering Services, Inc., 2004. Preliminary Geotechnical Engineering Investigation, Proposed Single-Family Residential Subdivision Tract No. 5368, Polk Avenue, Fresno, California. Prepared for Highland Partners Group, Inc.

_______, 2006. Preliminary Geotechnical Investigation Report, Proposed Tract 5400 Residential Subdivisions. SEC of Hayes and Gettysburg Avenues, Fresno, California. Prepared for Lennar Fresno, Inc.

_______, 2007. Preliminary Geotechnical Investigation Report, Proposed Single-Family Residential Subdivision Tract No. 5600. Grantland and Barstow Avenues, Fresno, California. Prepared for the McCaffrey Group, Inc.

The Twining Laboratories, Inc., 1991. Geotechnical Engineering Investigation. Tentative Tract 4282 and 4385. West Dakota Avenue and North Polk Avenue. Fresno, CA. Prepared for Monte Vista Development.

Unruh, J.R. and Moores, E.M., 1992. Quaternary blind thrusting in the southwestern Sacramento Valley

URS/HMM/Arup Joint Ventura, 2010. Geologic and Seismic Hazard Report: Fresno to Bakerfield Section. California High Speed Train Project.

________, 2011. Geology, Soils and Seismicity Technical Report: Fresno to Bakerfield Section. California High Speed Train Project.

U. S. Department of Agriculture (USDA) and Natural Resources Conservation Service (NRCS), 2008. *Soil Survey Geographic (SSURGO) database for Eastern Fresno Area County, California.* Fort Worth, Texas. Electronic document, available at: http://SoilDataMart.nrcs.usda.gov.

U.S. Federal Highway Administration, 2010. Drilled Shafts: Construction Procedures and LRFD Design Methods. FHWA-NHI-10-016.

U.S. Geological Survey (USGS), 2005. Preliminary integrated databases for the United States - Western States: California, Nevada, Arizona, and Washington: U.S. Geological Survey Open-File Report OFR 2005-1305, U.S. Geological Survey, Reston, Virginia, USA.

________, 2006. Quaternary Fault and Fold Database for the United States, Electronic document, available at: http//earthquakes.usgs.gov/regional/qfaults/.

________, 2008. Documentation for the 2008 Update of the United States National Seismic Hazard Maps.

Youd T. L. et al., 2001. Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils J. Geotech. and Geoenvir. Engrg. Volume 127, Issue 10, pp. 817-833.

Appendix B – Fresno Grade Separation

SEISMIC ANALYSIS AND DESIGN PLAN

General Classification

As this structure directly supports the HST track it is designated a **Primary Structure** in accordance with TM 2.10.4.

Importance Classification

The structure lies on the main route north of Fresno Station therefore in accordance with TM 2.10.4 cl 6.5.1.2 it is proposed to be designated an Important Structure.

Technical Classification

The structure does not conform to the requirements of a Standard Structure. Neither does it possess any of the features that might class it as a complex structure.

Therefore in accordance with TM 2.10.4 cl 6.5.1.3 it is proposed to be designated a Non-Standard Structure.

Analysis

The overall structure is a U-trough which in different locations is un-braced, braced or has a cover slab.

Cross sections of the U-Trough have been designed as reinforced concrete structures using the LRFD code with Caltrans amendments if applicable. Loading for these sections have been developed assuming that they are rigid walls in accordance with TM 2.3.2. Additional pressure has been applied to represent force that may be locked into the structure through the temporary excavation bracing.

Live Load Surcharge to represent Cooper E80 train loading or adjacent development surcharge has been applied as appropriate.

Additional seismic earth pressures are not applied as the TM 2.9.10 does not require additional pressures where the PGA is less than 0.35g.

As the U-Trough is, in general, symmetrical no calculations have been presented to demonstrate stability from sliding etc.

Consideration of buoyancy has been made and calculations made to demonstrate the Factor of Safety against flotation or the additional tie down force that may be required if FoS is inadequate.

For 30% design the calculations have concentrated on proving the overall section adequacy and reinforcement capacity.

U-Trough Calculations

INTRODUCTION

The following calculations represent a set of preliminary sizing calculations for the **Fresno Grade Separation**, which is a concrete **U-Trough** structure 8500 feet in length (almost 2 miles).
The **U-Trough** is a linear feature whose design is mainly concerned with the properties of cross sections taken at intervals along its length. These cross sections have a general similarity but with certain elements varying from section to section depending on changing conditions at each location.
Each section considered has a number of basic parameters that are considered to be invariant throughout the structure together with a number of other parameters that vary according to the particular station at which the section is drawn.

Parameters that are considered invariant include:
Applicable load factors for design
Concrete grade
Reinforcement Grade
Soil properties (density, cohesion and angle of internal friction)

Parameters that may vary according to station are:
Ground Level
Top of Rail Level
Ground Water Level
Base Slab Thickness
Wall thickness
Roof slab thickness (covered sections)
Width of the trough (from the inner face of left wall to inner face of right wall)
Presence of a crash wall
Whether the section is covered
Whether the section is braced and spacing of braces
The additional surcharge applied to the ground surface adjacent to the trough
The values used in these calculations are listed on the following pages.

GEOMETRICAL PARAMETERS

Trench start station ... 1088500.000 ft
Trench end station ... 1097000.000 ft
Section check interval ... 50.000 ft
Trench wall thickness ... 3.000 ft
Vertical distance from top of rail level to top of base slab ... 2.500 ft
Vertical distance from top of rail level to standard roof cover ... 27.000 ft
Normal roof slab thickness ... 3.000 ft
Jacked box section thickness (walls, base and roof) ... 5.000 ft

LOADING PARAMETERS

The primary loading parameter is simply the earth pressure of the ground in which the trench is constructed, including the level of groundwater in the soil. Additionally the design takes account of the possibility of loads applied adjacent to the trench.
These may be from:
Future maintenance works alongside the trench;
The loads imposed by UPRR trains;
Accidental surcharge due to a derailment.

ALIGNMENT DETAILS

Alignment Report

No Horizontal Data
Vertical Curve from 1087882.080, 299.830, 0.001100 to 1089882.080, 281.930, -0.019000
Vertex at Sta 1087991.533, 299.890
Vertical IP at Sta 1088882.080, 300.930
Parameters (y=a.x^2+b.x+c) a= -5.025000e-06, b= 1.093431e+01, c= -5947921.125713

Vertical Grade from 1089882.080, 281.930 to 1090875.480, 263.060
Parameters (z=ax+b) a= -0.018995, b= 20984.643

Vertical Curve from 1090875.480, 263.060, -0.019000 to 1094175.480, 257.280, 0.015500
Vertex at Sta 1092692.871, 245.795
Vertical IP at Sta 1092525.625, 231.707
Parameters (y=a.x^2+b.x+c) a= 5.227273e-06, b= -1.142361e+01, c= 6241492.920461

Vertical Grade from 1094175.480, 257.280 to 1095035.980, 270.620
Parameters (z=ax+b) a=   0.015503, b= -16705.301

Vertical Curve from 1095035.980, 270.620, 0.015500 to 1096735.980, 285.920, 0.002500
Vertex at Sta 1097062.903, 286.329
Vertical IP at Sta 1095885.980, 283.795
Parameters (y=a.x^2+b.x+c) a= -3.823529e-06, b= 8.389305e+00, c= -4601511.075169

Vertical Grade from 1096735.980, 285.920 to 1097500.010, 287.830
Parameters (z=ax+b) a=   0.002500, b= -2455.884

STRUCTURE FORM PARAMETERS

The trench includes a number of changes in structural form. The transition from one form to another is governed by external features or simply by depth below original or final ground level.
Covered Section at Station ... 1092020.000 ft, to Station 1092205.000 ft
Covered Section at Station ... 1093375.000 ft, to Station 1093520.000 ft
Covered Section at Station ... 1093695.000 ft, to Station 1093960.000 ft
Special stations included at ... 1090285.000 ft
Special stations included at ... 1093402.000 ft
Special stations included at ... 1093425.000 ft
Special stations included at ... 1093510.000 ft
Special stations included at ... 1093625.000 ft
Special stations included at ... 1093975.000 ft

GEOTECHNICAL PARAMETERS

Earth Pressure coefficients have been calculated from standard Rankine formulae based on parameters recommended in the Preliminary Design Memorandum.
Soil bulk density 125.000 Pcf
Soil angle of internal friction   33.400 Deg
Soil cohesion   0.000 Psf

Resulting soil pressure parameters
Active earth pressure coefficient   0.290
"At-Rest" earth pressure coefficient   0.450
Passive earth pressure coefficient (not used)   3.449

STRUCTURAL PARAMETERS

Structural parameters based on sections size etc are determined within the calculations, but the basic material properties used areas stated below.
Concrete strength grade used in calculations   5.0 psi
Reinforcement strength grade used in calculations   60.0 psi

ORIGINAL GROUND LEVELS
These have been extracted from the project survey and are located at the centre of the trough.

Point 1087500, 296.82
Point 1087550, 296.68
Point 1087600, 296.52
Point 1087650, 296.59
Point 1087700, 296.83
Point 1087750, 296.89
Point 1087800, 297.1
Point 1087850, 297.3
Point 1087882.08, 297.32
Point 1087900, 297.35
Point 1087950, 297.38
Point 1088000, 297.13
Point 1088050, 297.04
Point 1088100, 297
Point 1088150, 296.77
Point 1088200, 296.71
Point 1088250, 296.58
Point 1088300, 296.25
Point 1088350, 296.13
Point 1088400, 295.88
Point 1088450, 295.75
Point 1088500, 295.59
Point 1088550, 295.37
Point 1088600, 295.32
Point 1088650, 295.15
Point 1088700, 294.9
Point 1088750, 294.77
Point 1088800, 294.61
Point 1088850, 294.45
Point 1088900, 294.34
Point 1088950, 294.27
Point 1089000, 294.12
Point 1089050, 293.99
Point 1089100, 293.89
Point 1089150, 293.71
Point 1089200, 293.53
Point 1089250, 293.37
Point 1089300, 293.27
Point 1089350, 293.17
Point 1089400, 293.02
Point 1089450, 292.8
Point 1089500, 292.57
Point 1089550, 292.44
Point 1089600, 292.32
Point 1089650, 292.2
Point 1089700, 291.97
Point 1089750, 291.81
Point 1089800, 291.69
Point 1089850, 291.52
Point 1089882.08, 291.42
Point 1089900, 291.36
Point 1089950, 291.18
Point 1090000, 290.99
Point 1090050, 290.81
Point 1090100, 290.74
Point 1090150, 290.62
Point 1090200, 290.62
Point 1090250, 290.61
Point 1090300, 290.62
Point 1090350, 290.34
Point 1090400, 289.71
Point 1090450, 288.92
Point 1090500, 288.99
Point 1090550, 288.85
Point 1090600, 288.7
Point 1090650, 288.77
Point 1090700, 288.73
Point 1090750, 288.95
Point 1090800, 288.77
Point 1090850, 288.66
Point 1090875.48, 288.64
Point 1090900, 288.63
Point 1090950, 288.37
Point 1091000, 286.04
Point 1091050, 275.96
Point 1091100, 284.24
Point 1091150, 288.49
Point 1091200, 287.59
Point 1091250, 287.47
Point 1091300, 287.45
Point 1091350, 286.76
Point 1091400, 287.4
Point 1091450, 286.96
Point 1091500, 287.43
Point 1091550, 287.77
Point 1091600, 288.03
Point 1091650, 288.36
Point 1091700, 288.89
Point 1091750, 289.6
Point 1091800, 289.74
Point 1091850, 290.37
Point 1091900, 292.28
Point 1091950, 289.75
Point 1092000, 289.33
Point 1092050, 289.01
Point 1092100, 288.35
Point 1092150, 288.21
Point 1092200, 289.39
Point 1092250, 288.64
Point 1092300, 287.74
Point 1092350, 287.17
Point 1092400, 288.43
Point 1092450, 288.54
Point 1092500, 288.51
Point 1092550, 288.28
Point 1092600, 288.16
Point 1092650, 287.9
Point 1092700, 287.57
Point 1092750, 286.85
Point 1092800, 286.61
Point 1092850, 286.86
Point 1092900, 287.28
Point 1092950, 287.71
Point 1093000, 287.97
Point 1093050, 288.08
Point 1093100, 289.68
Point 1093150, 290.7
Point 1093200, 290.04
Point 1093250, 290.53
Point 1093300, 290.09
Point 1093350, 287.15
Point 1093400, 285.04
Point 1093450, 287.34
Point 1093500, 286.45
Point 1093550, 288.23
Point 1093600, 287.99
Point 1093650, 287.51
Point 1093700, 301.48
Point 1093750, 317.56
Point 1093800, 318.65
Point 1093850, 318.51
Point 1093900, 317.77
Point 1093950, 308.11
Point 1094000, 288.76
Point 1094050, 288.74
Point 1094100, 289.07
Point 1094150, 288.88
Point 1094175.48, 288.77
Point 1094200, 288.92
Point 1094250, 289.15
Point 1094300, 289.16
Point 1094350, 288.01
Point 1094400, 287.57
Point 1094450, 287.56
Point 1094500, 287.52
Point 1094550, 286.49
Point 1094600, 288.13
Point 1094650, 287.75
Point 1094700, 287.83
Point 1094750, 287.8
Point 1094800, 287.91
Point 1094850, 286.6
Point 1094900, 287.23
Point 1094950, 286.65
Point 1095000, 286.02
Point 1095035.98, 285.85
Point 1095050, 285.78
Point 1095100, 285.9
Point 1095150, 286.13
Point 1095200, 282.99
Point 1095250, 286.45
Point 1095300, 286.86
Point 1095350, 286.92
Point 1095400, 286.8
Point 1095450, 286.77
Point 1095500, 286.79
Point 1095550, 286.82
Point 1095600, 286.64
Point 1095650, 286.38
Point 1095700, 287.47
Point 1095750, 287.64
Point 1095800, 287.75
Point 1095850, 287.72
Point 1095900, 286.9
Point 1095950, 286.34
Point 1096000, 286.81
Point 1096050, 286.75
Point 1096100, 286.62
Point 1096150, 286.22
Point 1096200, 286.0
Point 1096250, 286.17
Point 1096300, 286.31
Point 1096350, 286.02
Point 1096400, 286.45
Point 1096450, 286.74
Point 1096500, 286.64
Point 1096550, 287.02
Point 1096600, 290.02
Point 1096650, 286.28
Point 1096700, 286.47
Point 1096735.98, 286.31
Point 1096750, 286.26
Point 1096800, 286.18
Point 1096850, 286.45
Point 1096900, 286.72
Point 1096950, 287.02
Point 1097000, 286.96
Point 1097050, 286.3
Point 1097100, 285.95
Point 1097150, 285.73
Point 1097200, 285.51
Point 1097250, 285.49
Point 1097300, 284.98
Point 1097350, 285.4
Point 1097400, 285.03
Point 1097450, 285.88
Point 1097500, 285.79
Point 1097550, 285.8

FINAL GROUND LEVELS
Where they may be higher than the original ground level (OGL).

Point 1088500.0, 0.0
Point 1090500.0, 0.0
Point 1091000.0, 290.0
Point 1091500.0, 290.0
Point 1091550.0, 0.0
Point 1093300.0, 0.0
Point 1093350.0, 289.0
Point 1093550.0, 289.0
Point 1093600.0, 0.0
Point 1097550.0, 0.0

GROUNDWATER LEVELS
No reliable measured groundwater levels exist at present as there are not yet sufficient datapoints to provide the necessary confidence. The levels used in these calculations are based on the guidance provided in the Geotechnical Design Memorandum.

Point 1088500.0, 235.0
Point 1090500.0, 234.0
Point 1091100.0, 280.0
Point 1091500.0, 280.0
Point 1091750.0, 235.0
Point 1093000.0, 235.0
Point 1093300.0, 279.0
Point 1093699.0, 279.0
Point 1093700.0, 235.0
Point 1097510.0, 235.0

BASE THICKNESS
Stationing indicates where thickness changes from previous value. Where high groundwater may result in positive buoyancy. The base thickness is not increased, but minimum tie down force is indicated. This may be provided by additional thickness or by the use of tension piles.

Point 1088500.0, 5.0
Point 1097510.0, 3.0

HEIGHT OF COLLISION WALL
Stationing indicates where height changes from previous value.

Point 1088500.0, 10.0
Point 1095020.0, 3.0
Point 1097510.0, 3.0

WALL BRACE SPACING (IF REQUIRED)
Stationing indicates where spacing changes from previous value.

Point 1088500.0, 20.0
Point 1091550.0, 10.0
Point 1094100.0, 20.0
Point 1097510.0, 20.0

WIDTH OF U-TROUGH
Stationing indicates where the trough width (between internal faces) changes from the previous value.

Point 1088500.0, 42.0
Point 1093694.0, 42.0
Point 1093695.0, 43.0
Point 1093960.0, 43.0
Point 1093961.0, 42.0
Point 1097000.0, 42.0
Point 1097100.0, 42.15
Point 1097200.0, 43.0
Point 1097300.0, 45.0
Point 1097400.0, 48.18
Point 1097500.0, 52.54

SURCHARGE PRESSURE ADJACENT TO U-TROUGH
Stationing indicates where surcharge changes from previous value.
The Surcharge of 420Psf is equivalent to a Cooper E80 train at 20ft from the trough wall. The UP lines are always further from the HST ROW so that this value is conservative. Where the SJVR track crosses the covered section of the trough, the full Cooper E80 surcharge (1882 Psf) is applied over the whole span. This is also conservative.
Surcharge is assumed to be applied to both sides of trough.

Point 1088500.0, 420.0
Point 1092020.0, 1882.0
Point 1092205.0, 420.0
Point 1093375.0, 1882.0
Point 1093520.0, 420.0
Point 1093695.0, 0.0
Point 1093960.0, 600.0
Point 1094100.0, 600.0
Point 1097510.0, 100.0

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10885+ 0.000
Original Ground Level 295.590
Groundwater Level 235.000
Top of Rail 298.591
Top of Base 296.091
Founding Level 291.091

UN-BRACED U-TROUGH
Trough Depth = -0.501 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf

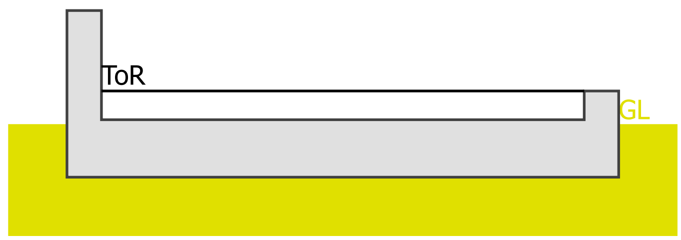
WALL ROOT SECTION at -0.501 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 260.349 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.938	2.500
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 2.977e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 275.257 kip-ft/ft
Shear Checks
Required capacity = 520.084 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 569.089 kip



Fresno Grade Separation
Preliminary Design

Section at 10885+50.000
Original Ground Level 295.370
Groundwater Level 234.975
Top of Rail 298.323
Top of Base 295.823
Founding Level 290.823

UN-BRACED U-TROUGH
Trough Depth = -0.453 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf

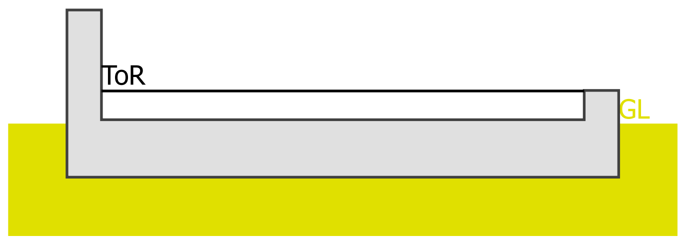
WALL ROOT SECTION at -0.453 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 260.330 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.938	2.500
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 2.977e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 275.257 kip-ft/ft
Shear Checks
Required capacity = 520.120 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 569.089 kip



California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10886+ 0.000
Original Ground Level 295.320
Groundwater Level 234.950
Top of Rail 298.030
Top of Base 295.530
Founding Level 290.530

UN-BRACED U-TROUGH
Trough Depth = -0.210 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf

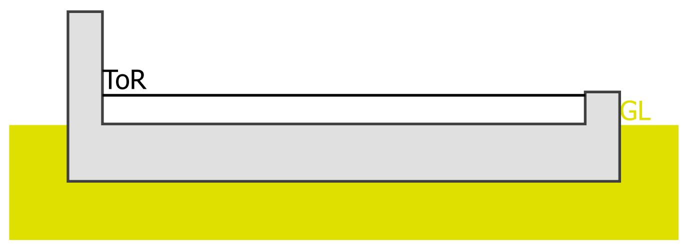
WALL ROOT SECTION at -0.210 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 260.263 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.938	2.500
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 2.977e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 275.257 kip-ft/ft
Shear Checks
Required capacity = 520.313 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 569.089 kip

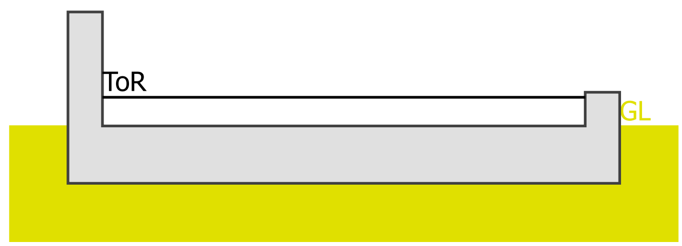


Section at 10886+50.000
Original Ground Level 295.150
Groundwater Level 234.925
Top of Rail 297.711
Top of Base 295.211
Founding Level 290.211

UN-BRACED U-TROUGH
Trough Depth = -0.061 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at -0.061 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 260.246 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.938	2.500
Area (sq-in)	1.988	0.000

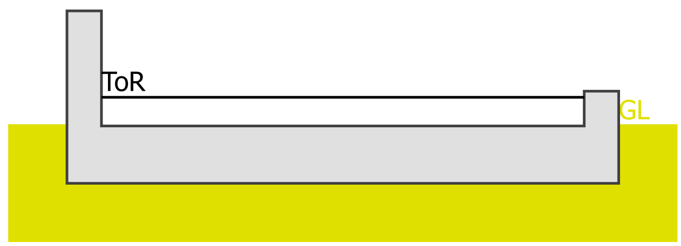
Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 2.977e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 275.257 kip-ft/ft
Shear Checks
Required capacity = 520.436 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 569.089 kip

Section at 10887+ 0.000
Original Ground Level 294.900
Groundwater Level 234.900
Top of Rail 297.368
Top of Base 294.868
Founding Level 289.868

UN-BRACED U-TROUGH
Trough Depth = 0.032 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 0.032 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 260.244 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

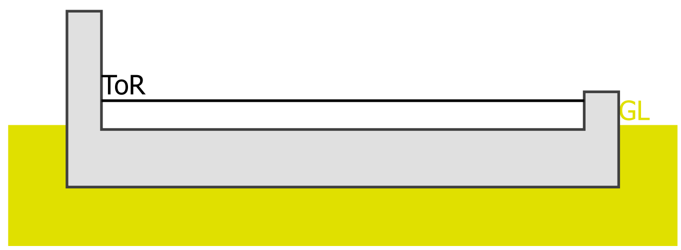
Layer	Tension	Compression
Depth (in)	31.938	2.500
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 2.977e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 275.257 kip-ft/ft
Shear Checks
Required capacity = 520.515 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 569.089 kip

Section at 10887+50.000
Original Ground Level 294.770
Groundwater Level 234.875
Top of Rail 296.999
Top of Base 294.499
Founding Level 289.499

UN-BRACED U-TROUGH
Trough Depth = 0.271 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf

WALL ROOT SECTION at 0.271 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 260.275 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.12 0.00 0.00
Smaller Bar 1.12 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 31.938 2.500
Area (sq-in) 1.988 0.000
Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 2.977e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 275.257 kip-ft/ft
Shear Checks
Required capacity = 520.725 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 569.089 kip



Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10888+ 0.000
Original Ground Level 294.610
Groundwater Level 234.850
Top of Rail 296.606
Top of Base 294.106
Founding Level 289.106

UN-BRACED U-TROUGH
Trough Depth = 0.504 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf

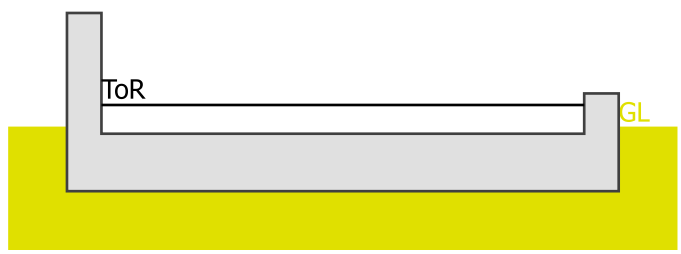
WALL ROOT SECTION at 0.504 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 260.354 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.938	2.500
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 2.977e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 275.257 kip-ft/ft
Shear Checks
Required capacity = 520.939 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 569.089 kip



Section at 10888+50.000
Original Ground Level 294.450
Groundwater Level 234.825
Top of Rail 296.187
Top of Base 293.687
Founding Level 288.687

UN-BRACED U-TROUGH
Trough Depth = 0.763 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf

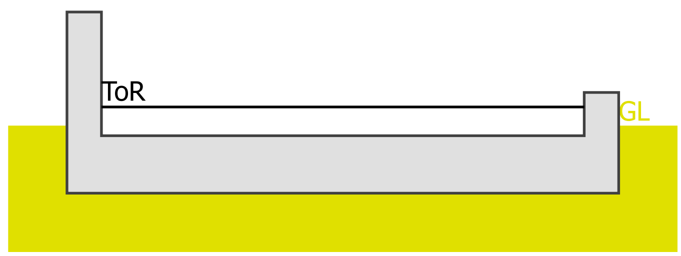
WALL ROOT SECTION at 0.763 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 260.498 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.938	2.500
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 2.977e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 275.257 kip-ft/ft
Shear Checks
Required capacity = 521.188 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 569.089 kip



California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10889+ 0.000
Original Ground Level 294.340
Groundwater Level 234.800
Top of Rail 295.743
Top of Base 293.243
Founding Level 288.243

UN-BRACED U-TROUGH
Trough Depth = 1.097 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf

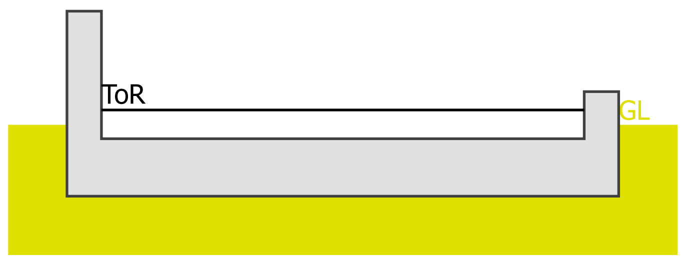
WALL ROOT SECTION at 1.097 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 260.775 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.938	2.500
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 2.977e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 275.257 kip-ft/ft
Shear Checks
Required capacity = 521.527 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 569.089 kip



Fresno Grade Separation
Preliminary Design

Section at 10889+50.000
Original Ground Level 294.270
Groundwater Level 234.775
Top of Rail 295.274
Top of Base 292.774
Founding Level 287.774

UN-BRACED U-TROUGH
Trough Depth = 1.496 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf

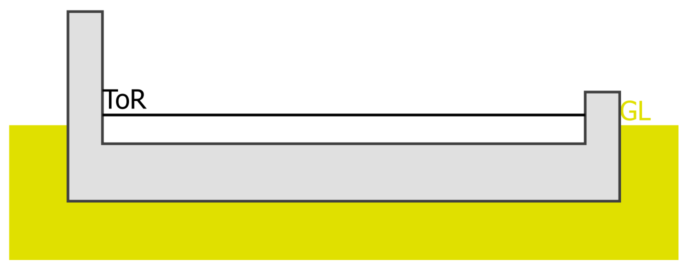
WALL ROOT SECTION at 1.496 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 261.245 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.938	2.500
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 2.977e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 275.257 kip-ft/ft
Shear Checks
Required capacity = 521.957 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 569.089 kip



California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10890+ 0.000
Original Ground Level 294.120
Groundwater Level 234.750
Top of Rail 294.780
Top of Base 292.280
Founding Level 287.280

UN-BRACED U-TROUGH
Trough Depth = 1.840 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf

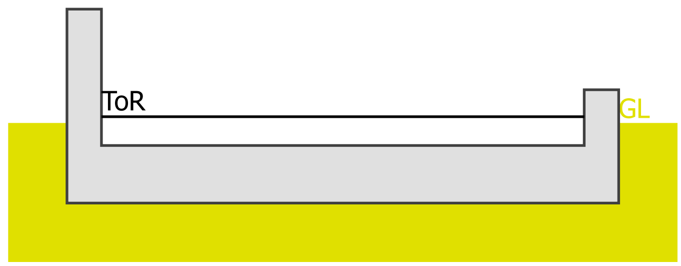
WALL ROOT SECTION at 1.840 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 261.776 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.938	2.500
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 2.977e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 275.257 kip-ft/ft
Shear Checks
Required capacity = 522.351 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 569.089 kip



Fresno Grade Separation
Preliminary Design

Section at 10890+50.000
Original Ground Level 293.990
Groundwater Level 234.725
Top of Rail 294.260
Top of Base 291.760
Founding Level 286.760

UN-BRACED U-TROUGH
Trough Depth = 2.230 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf

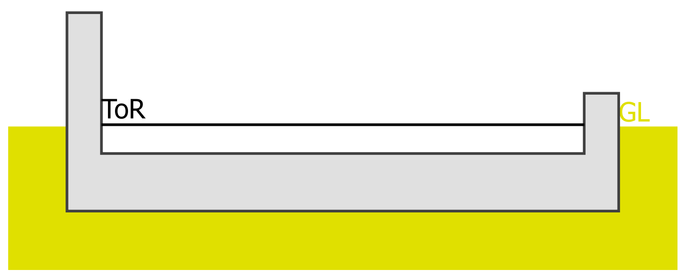
WALL ROOT SECTION at 2.230 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 262.521 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.938	2.500
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 2.977e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 275.257 kip-ft/ft
Shear Checks
Required capacity = 522.821 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 569.089 kip



Section at 10891+ 0.000
Original Ground Level 293.890
Groundwater Level 234.700
Top of Rail 293.716
Top of Base 291.216
Founding Level 286.216

UN-BRACED U-TROUGH
Trough Depth = 2.674 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf

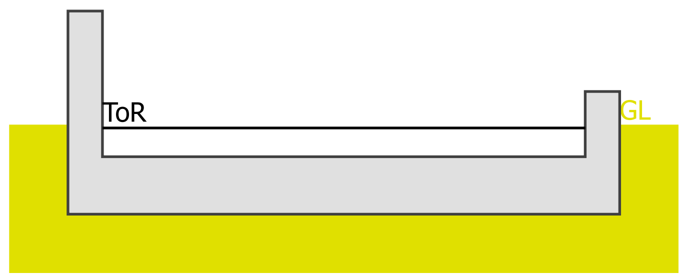
WALL ROOT SECTION at 2.674 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 263.566 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.938	2.500
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 2.977e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 275.257 kip-ft/ft
Shear Checks
Required capacity = 523.391 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 569.089 kip



California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10891+50.000
Original Ground Level 293.710
Groundwater Level 234.675
Top of Rail 293.146
Top of Base 290.646
Founding Level 285.646

UN-BRACED U-TROUGH
Trough Depth = 3.064 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf

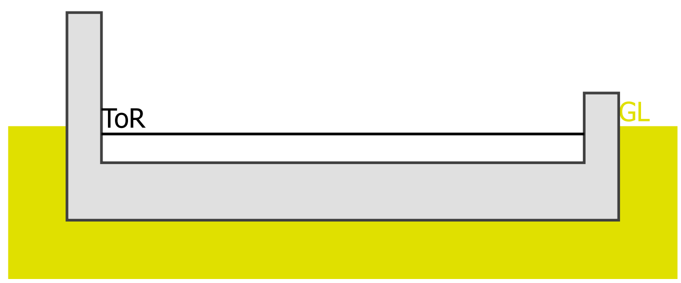
WALL ROOT SECTION at 3.064 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 264.658 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.938	2.500
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 2.977e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 275.257 kip-ft/ft
Shear Checks
Required capacity = 523.919 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 569.089 kip



Fresno Grade Separation
Preliminary Design

Section at 10892+ 0.000
Original Ground Level 293.530
Groundwater Level 234.650
Top of Rail 292.552
Top of Base 290.052
Founding Level 285.052

UN-BRACED U-TROUGH
Trough Depth = 3.478 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf

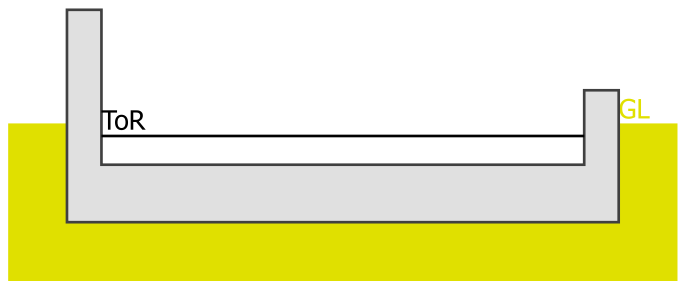
WALL ROOT SECTION at 3.478 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 266.007 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.938	2.500
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 2.977e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 275.257 kip-ft/ft
Shear Checks
Required capacity = 524.510 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 569.089 kip



Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10892+50.000
Original Ground Level 293.370
Groundwater Level 234.625
Top of Rail 291.932
Top of Base 289.432
Founding Level 284.432

UN-BRACED U-TROUGH
Trough Depth = 3.938 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf

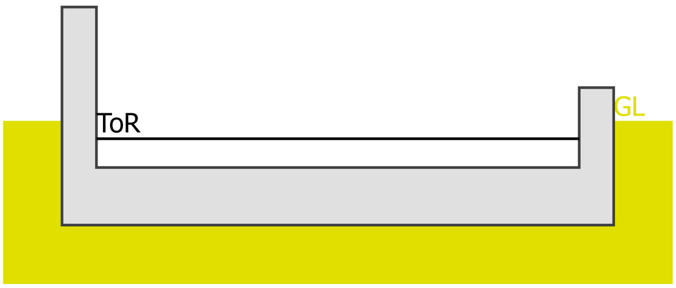
WALL ROOT SECTION at 3.938 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 267.736 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.938	2.500
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 2.977e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 275.257 kip-ft/ft
Shear Checks
Required capacity = 525.200 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 569.089 kip



Fresno Grade Separation Preliminary Design

Section at 10893+ 0.000
Original Ground Level 293.270
Groundwater Level 234.600
Top of Rail 291.287
Top of Base 288.787
Founding Level 283.787

UN-BRACED U-TROUGH
Trough Depth = 4.483 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf

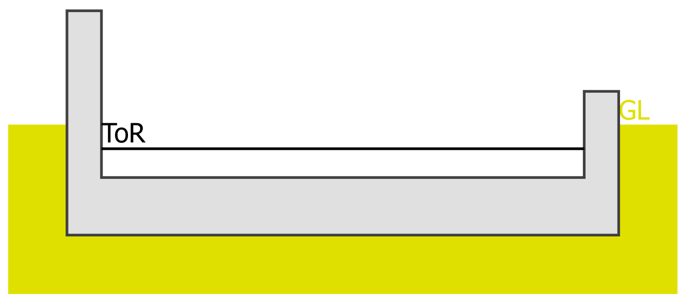
WALL ROOT SECTION at 4.483 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 270.113 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.938	2.500
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 2.977e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 275.257 kip-ft/ft
Shear Checks
Required capacity = 526.067 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 569.089 kip



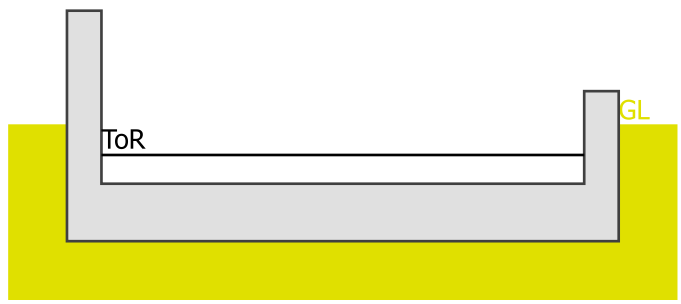
Fresno Grade Separation
Preliminary Design

Section at 10893+50.000
Original Ground Level 293.170
Groundwater Level 234.575
Top of Rail 290.617
Top of Base 288.117
Founding Level 283.117

UN-BRACED U-TROUGH
Trough Depth = 5.053 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 5.053 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 272.995 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.938	2.500
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 2.977e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 275.257 kip-ft/ft
Shear Checks
Required capacity = 527.030 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 569.089 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10894+ 0.000
Original Ground Level 293.020
Groundwater Level 234.550
Top of Rail 289.922
Top of Base 287.422
Founding Level 282.422

UN-BRACED U-TROUGH
Trough Depth = 5.598 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf

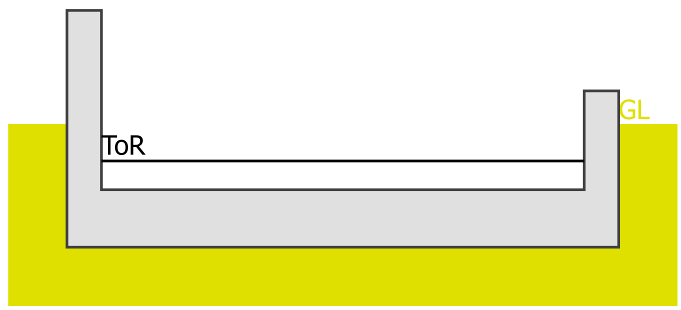
WALL ROOT SECTION at 5.598 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 276.145 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.875	2.500
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 528.003 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 569.089 kip



Fresno Grade Separation
Preliminary Design

Section at 10894+50.000
Original Ground Level 292.800
Groundwater Level 234.525
Top of Rail 289.201
Top of Base 286.701
Founding Level 281.701

UN-BRACED U-TROUGH
Trough Depth = 6.099 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf

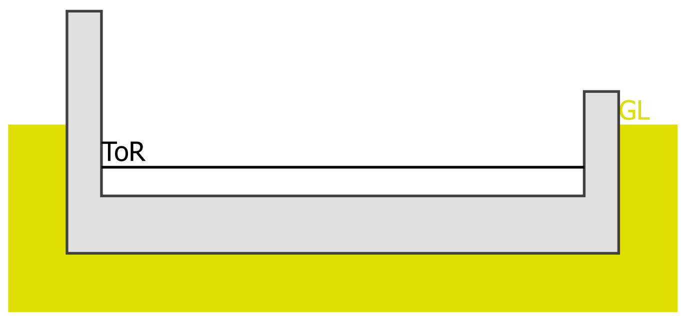
WALL ROOT SECTION at 6.099 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 279.387 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

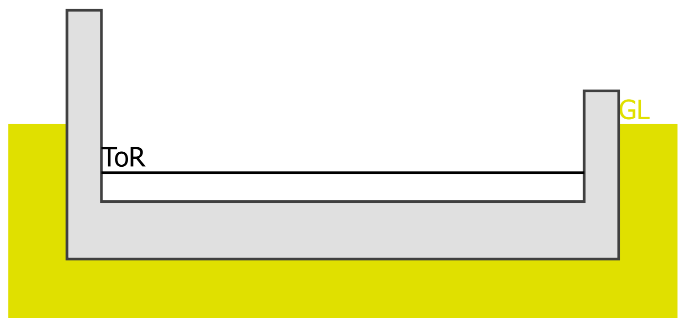
Layer	Tension	Compression
Depth (in)	31.875	2.500
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 528.943 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip



Section at 10895+ 0.000
Original Ground Level 292.570
Groundwater Level 234.500
Top of Rail 288.456
Top of Base 285.956
Founding Level 280.956

UN-BRACED U-TROUGH
Trough Depth = 6.614 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 6.614 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 283.089 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.875	2.500
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 529.957 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip

Section at 10895+50.000
Original Ground Level 292.440
Groundwater Level 234.475
Top of Rail 287.685
Top of Base 285.185
Founding Level 280.185

UN-BRACED U-TROUGH
Trough Depth = 7.255 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf

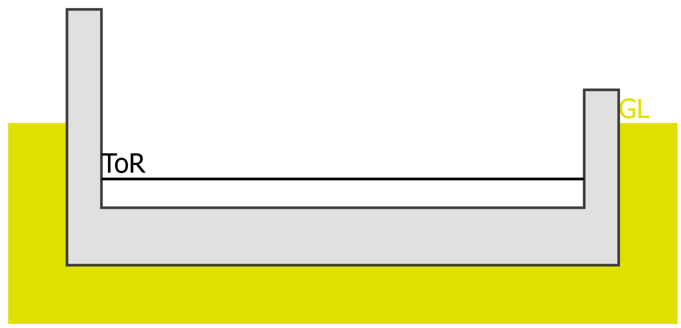
WALL ROOT SECTION at 7.255 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 288.222 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.875	2.500
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 531.282 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip



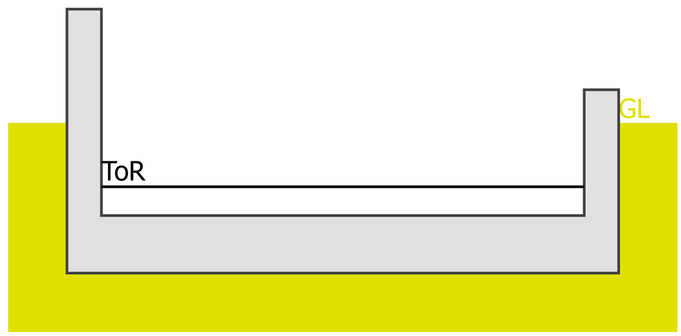
Fresno Grade Separation
Preliminary Design

Section at 10896+ 0.000
Original Ground Level 292.320
Groundwater Level 234.450
Top of Rail 286.890
Top of Base 284.390
Founding Level 279.390

UN-BRACED U-TROUGH
Trough Depth = 7.930 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 7.930 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 294.298 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.875	2.500
Area (sq-in)	2.454	0.000

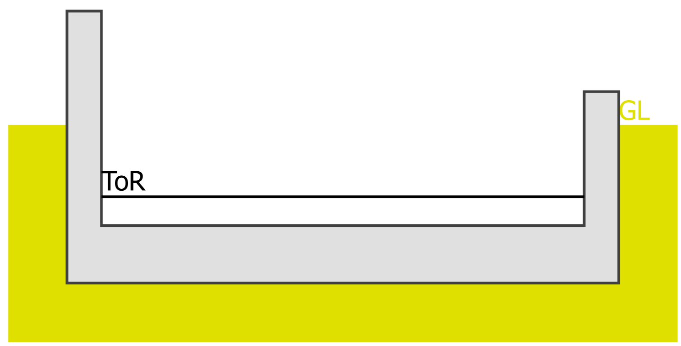
Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 532.758 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip

Section at 10896+50.000
Original Ground Level 292.200
Groundwater Level 234.425
Top of Rail 286.069
Top of Base 283.569
Founding Level 278.569

UN-BRACED U-TROUGH
Trough Depth = 8.631 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 8.631 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 301.347 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.875	2.500
Area (sq-in)	2.454	0.000

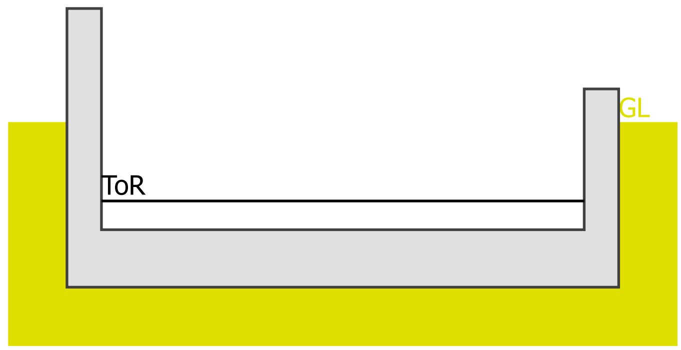
Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 534.373 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip

Section at 10897+ 0.000
Original Ground Level 291.970
Groundwater Level 234.400
Top of Rail 285.223
Top of Base 282.723
Founding Level 277.723

UN-BRACED U-TROUGH
Trough Depth = 9.247 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 9.247 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 308.194 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.875	2.500
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 535.865 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip

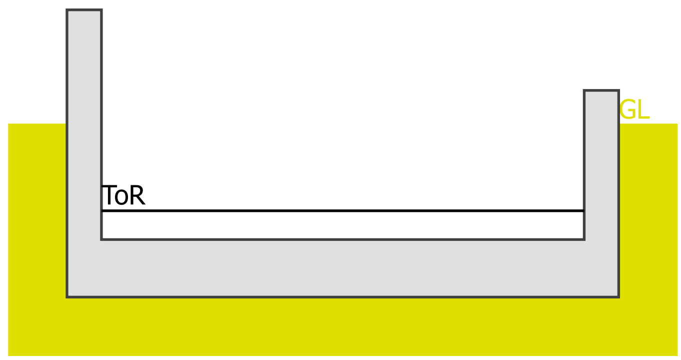
Fresno Grade Separation
Preliminary Design

Section at 10897+50.000
Original Ground Level 291.810
Groundwater Level 234.375
Top of Rail 284.352
Top of Base 281.852
Founding Level 276.852

UN-BRACED U-TROUGH
Trough Depth = 9.958 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 9.958 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 316.884 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.875	2.500
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 537.669 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip

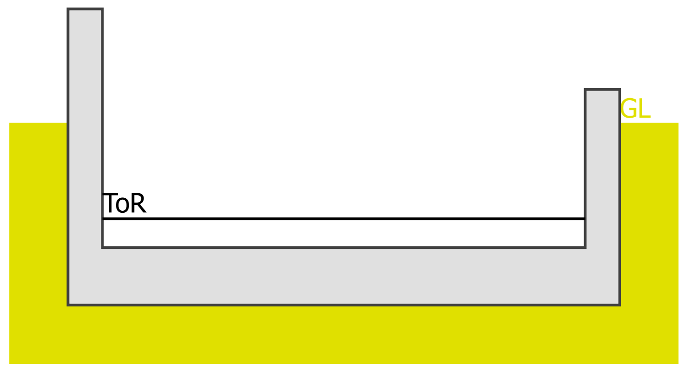
Fresno Grade Separation
Preliminary Design

Section at 10898+ 0.000
Original Ground Level 291.690
Groundwater Level 234.350
Top of Rail 283.456
Top of Base 280.956
Founding Level 275.956

UN-BRACED U-TROUGH
Trough Depth = 10.734 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 10.734 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 327.366 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.875	2.500
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 539.740 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 317.422 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

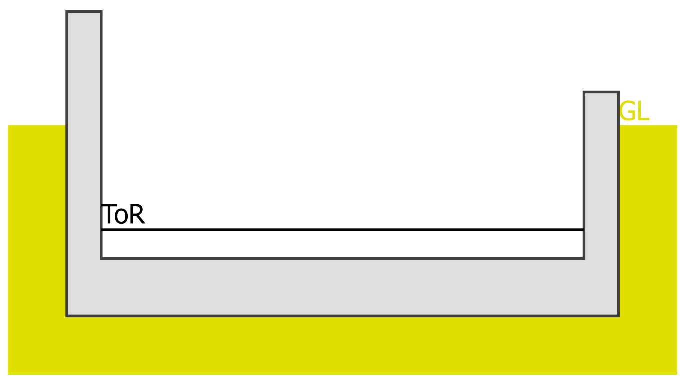
Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.875	2.500
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 537.778 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip

Fresno Grade Separation
Preliminary Design

Section at 10898+50.000
Original Ground Level 291.520
Groundwater Level 234.325
Top of Rail 282.534
Top of Base 280.034
Founding Level 275.034



UN-BRACED U-TROUGH
Trough Depth = 11.486 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf

WALL ROOT SECTION at 11.486 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 338.542 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.38	0.00	0.00
Smaller Bar	1.38	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.812	2.500
Area (sq-in)	2.970	0.000

Neutral Axis Depth = 4.367 in
Section is Tension controlled.
Reinforcement Strain 1.885e-02
Compression-block depth = 3.494 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 401.797 kip-ft/ft
Shear Checks
Required capacity = 541.845 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 317.422 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

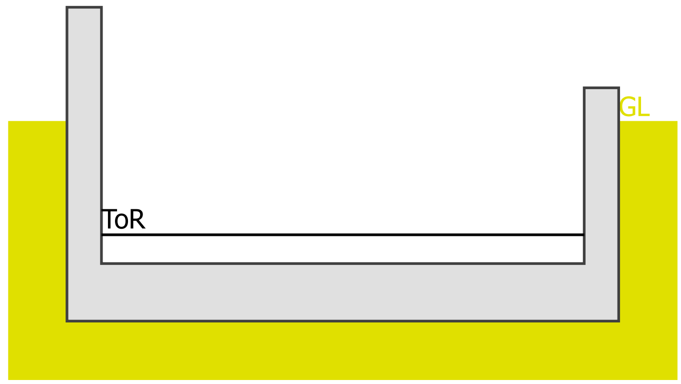
Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.875	2.500
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 537.778 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip

Section at 10899+ 0.000
Original Ground Level 291.360
Groundwater Level 234.300
Top of Rail 281.590
Top of Base 279.090
Founding Level 274.090

UN-BRACED U-TROUGH
Trough Depth = 12.270 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 12.270 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 351.336 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.38	0.00	0.00
Smaller Bar	1.38	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.812	2.500
Area (sq-in)	2.970	0.000

Neutral Axis Depth = 4.367 in
Section is Tension controlled.
Reinforcement Strain 1.885e-02
Compression-block depth = 3.494 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 401.797 kip-ft/ft
Shear Checks
Required capacity = 544.150 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 566.879 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 317.422 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

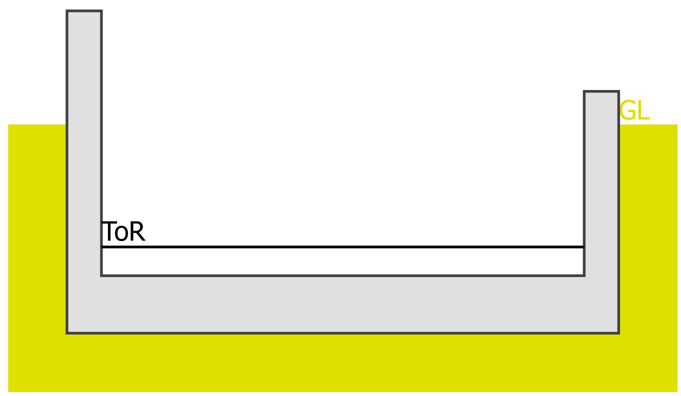
Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.875	2.500
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 537.778 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip

Section at 10899+50.000
Original Ground Level 291.180
Groundwater Level 234.275
Top of Rail 280.640
Top of Base 278.140
Founding Level 273.140

UN-BRACED U-TROUGH
Trough Depth = 13.040 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 13.040 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 365.039 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.38	0.00	0.00
Smaller Bar	1.38	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.812	2.500
Area (sq-in)	2.970	0.000

Neutral Axis Depth = 4.367 in
Section is Tension controlled.
Reinforcement Strain 1.885e-02
Compression-block depth = 3.494 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 401.797 kip-ft/ft
Shear Checks
Required capacity = 546.515 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 566.879 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 317.422 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

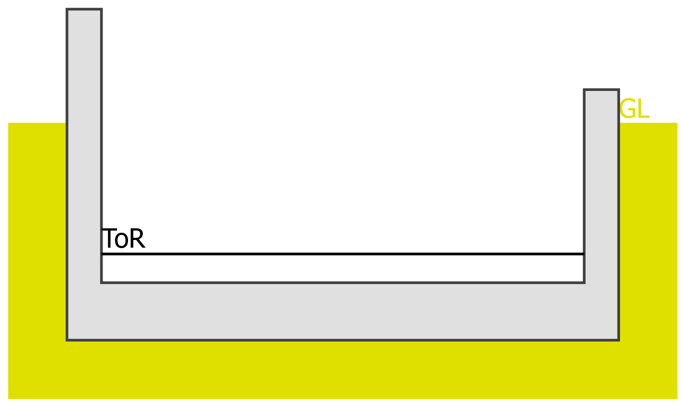
Layer	Tension	Compression
Depth (in)	31.875	2.500
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 537.778 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10900+ 0.000
Original Ground Level 290.990
Groundwater Level 234.250
Top of Rail 279.690
Top of Base 277.190
Founding Level 272.190



UN-BRACED U-TROUGH
Trough Depth = 13.800 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf

WALL ROOT SECTION at 13.800 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 379.724 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.38	0.00	0.00
Smaller Bar	1.38	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.812	2.500
Area (sq-in)	2.970	0.000

Neutral Axis Depth = 4.367 in
Section is Tension controlled.
Reinforcement Strain 1.885e-02
Compression-block depth = 3.494 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 401.797 kip-ft/ft
Shear Checks
Required capacity = 548.953 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 566.879 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 317.422 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

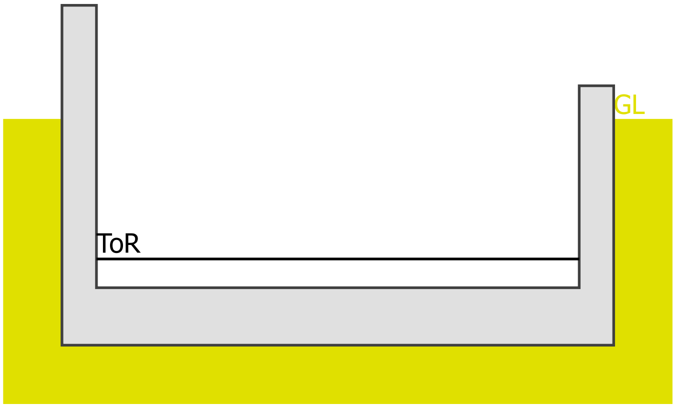
Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.875	2.500
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 537.778 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip

Section at 10900+50.000
Original Ground Level 290.810
Groundwater Level 234.225
Top of Rail 278.740
Top of Base 276.240
Founding Level 271.240

UN-BRACED U-TROUGH
Trough Depth = 14.570 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 14.570 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 395.817 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.38	0.00	0.00
Smaller Bar	1.38	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.812	2.500
Area (sq-in)	2.970	0.000

Neutral Axis Depth = 4.367 in
Section is Tension controlled.
Reinforcement Strain 1.885e-02
Compression-block depth = 3.494 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 401.797 kip-ft/ft
Shear Checks
Required capacity = 551.525 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 566.879 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 317.422 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.875	2.500
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 537.778 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip

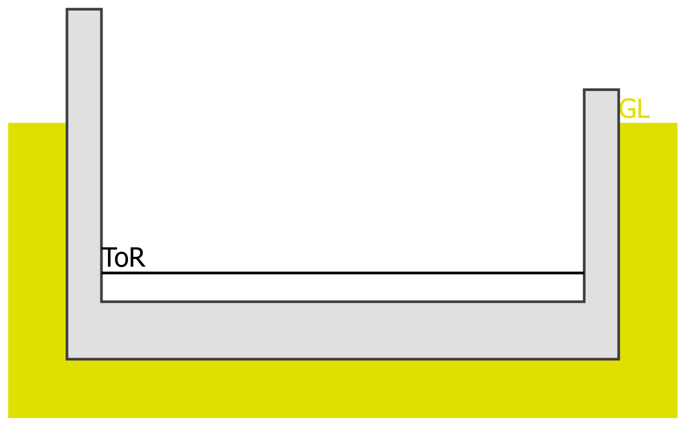
Fresno Grade Separation
Preliminary Design

Section at 10901+ 0.000
Original Ground Level 290.740
Groundwater Level 234.200
Top of Rail 277.791
Top of Base 275.291
Founding Level 270.291

UN-BRACED U-TROUGH
Trough Depth = 15.449 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 15.449 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 415.757 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.50	0.00	0.00
Smaller Bar	1.50	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.750	2.500
Area (sq-in)	3.534	0.000

Neutral Axis Depth = 5.197 in
Section is Tension controlled.
Reinforcement Strain 1.533e-02
Compression-block depth = 4.158 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 471.897 kip-ft/ft
Shear Checks
Required capacity = 554.593 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 565.774 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 317.422 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.875	2.500
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 537.778 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

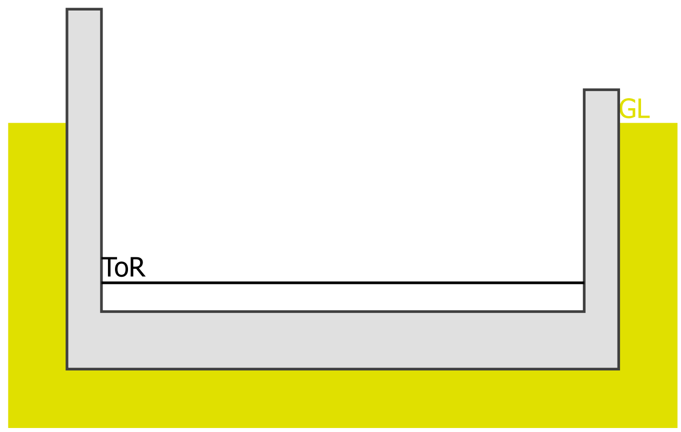
Fresno Grade Separation
Preliminary Design

Section at 10901+50.000
Original Ground Level 290.620
Groundwater Level 234.175
Top of Rail 276.841
Top of Base 274.341
Founding Level 269.341

UN-BRACED U-TROUGH
Trough Depth = 16.279 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 16.279 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 436.129 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.50	0.00	0.00
Smaller Bar	1.50	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.750	2.500
Area (sq-in)	3.534	0.000

Neutral Axis Depth = 5.197 in
Section is Tension controlled.
Reinforcement Strain 1.533e-02
Compression-block depth = 4.158 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 471.897 kip-ft/ft
Shear Checks
Required capacity = 557.611 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 565.774 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 317.422 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.875	2.500
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 537.778 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

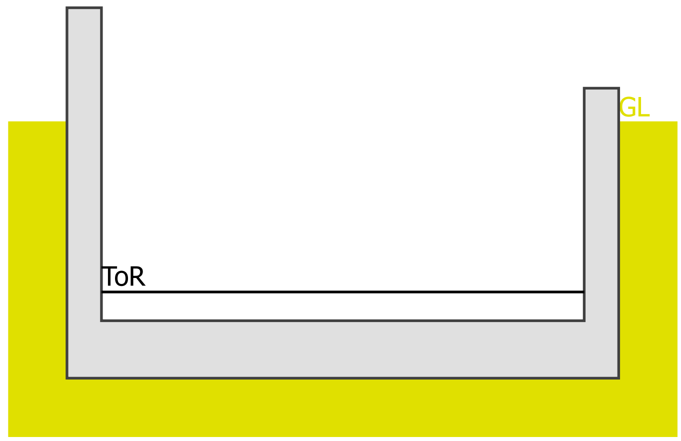
Fresno Grade Separation
Preliminary Design

Section at 10902+ 0.000
Original Ground Level 290.620
Groundwater Level 234.150
Top of Rail 275.891
Top of Base 273.391
Founding Level 268.391

UN-BRACED U-TROUGH
Trough Depth = 17.229 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 17.229 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 461.377 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.50	0.00	0.00
Smaller Bar	1.50	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.750	2.500
Area (sq-in)	3.534	0.000

Neutral Axis Depth = 5.197 in
Section is Tension controlled.
Reinforcement Strain 1.533e-02
Compression-block depth = 4.158 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 471.897 kip-ft/ft
Shear Checks
Required capacity = 561.214 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 565.774 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 317.422 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.875	2.500
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 537.778 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

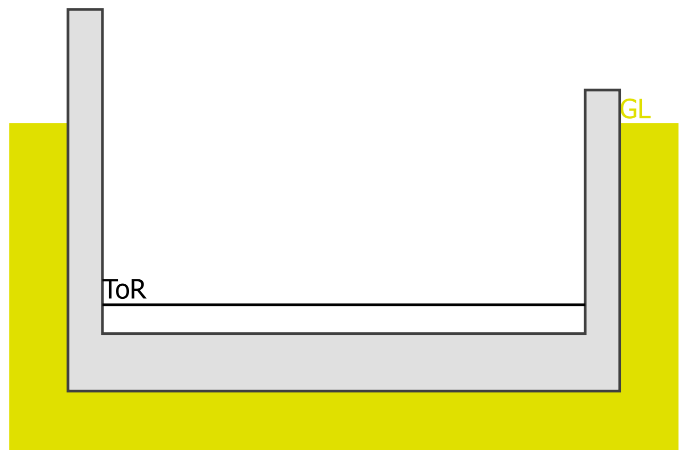
Fresno Grade Separation
Preliminary Design

Section at 10902+50.000
Original Ground Level 290.610
Groundwater Level 234.125
Top of Rail 274.941
Top of Base 272.441
Founding Level 267.441

UN-BRACED U-TROUGH
Trough Depth = 18.169 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 18.169 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 488.457 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.62	0.00	0.00
Smaller Bar	1.62	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.562	2.625
Area (sq-in)	4.148	0.000

Neutral Axis Depth = 6.100 in
Section is Tension controlled.
Reinforcement Strain 1.252e-02
Compression-block depth = 4.880 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 543.587 kip-ft/ft
Shear Checks
Required capacity = 564.934 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.625 in
Shear Reinforcement Area = 3.682 sq in/ft run
Shear Capacity Provided = 860.970 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 317.422 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.875	2.500
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 537.778 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip

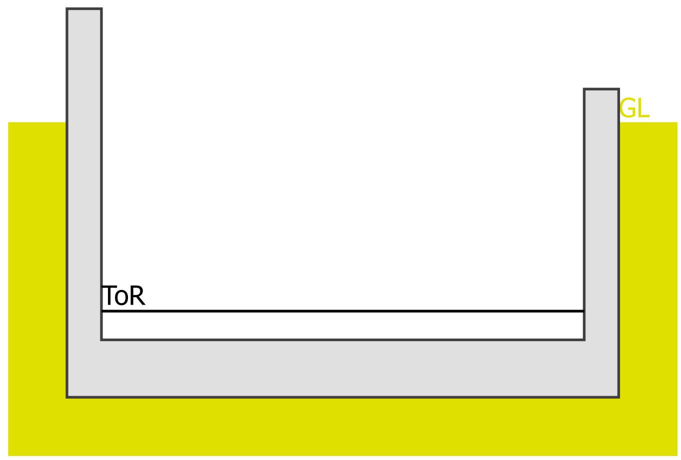
California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10902+85.000
Original Ground Level 290.617
Groundwater Level 234.107
Top of Rail 274.276
Top of Base 271.776
Founding Level 266.776

UN-BRACED U-TROUGH
Trough Depth = 18.841 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 18.841 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 509.137 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.62	0.00	0.00
Smaller Bar	1.62	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.562	2.625
Area (sq-in)	4.148	0.000

Neutral Axis Depth = 6.100 in
Section is Tension controlled.
Reinforcement Strain 1.252e-02
Compression-block depth = 4.880 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 543.587 kip-ft/ft
Shear Checks
Required capacity = 567.689 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.625 in
Shear Reinforcement Area = 3.682 sq in/ft run
Shear Capacity Provided = 860.970 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 317.422 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

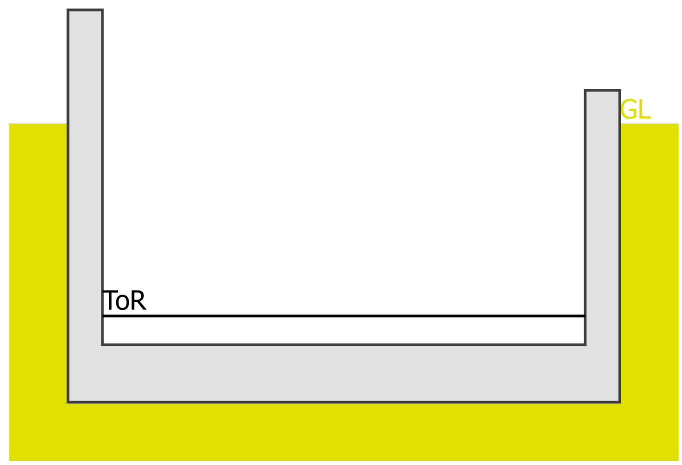
Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.875	2.500
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 537.778 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip

Section at 10903+ 0.000
Original Ground Level 290.620
Groundwater Level 234.100
Top of Rail 273.991
Top of Base 271.491
Founding Level 266.491

UN-BRACED U-TROUGH
Trough Depth = 19.129 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 19.129 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 518.345 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.62	0.00	0.00
Smaller Bar	1.62	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.562	2.625
Area (sq-in)	4.148	0.000

Neutral Axis Depth = 6.100 in
Section is Tension controlled.
Reinforcement Strain 1.252e-02
Compression-block depth = 4.880 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 543.587 kip-ft/ft
Shear Checks
Required capacity = 568.894 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.625 in
Shear Reinforcement Area = 3.682 sq in/ft run
Shear Capacity Provided = 860.970 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 317.422 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.875	2.500
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 537.778 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip

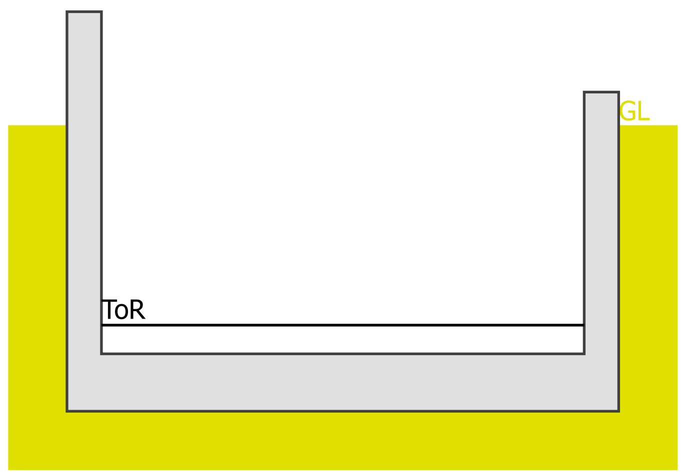
Fresno Grade Separation Preliminary Design

Section at 10903+50.000
Original Ground Level 290.340
Groundwater Level 234.075
Top of Rail 273.042
Top of Base 270.542
Founding Level 265.542

UN-BRACED U-TROUGH
Trough Depth = 19.798 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 19.798 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 540.578 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.62	0.00	0.00
Smaller Bar	1.62	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.562	2.625
Area (sq-in)	4.148	0.000

Neutral Axis Depth = 6.100 in
Section is Tension controlled.
Reinforcement Strain 1.252e-02
Compression-block depth = 4.880 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 543.587 kip-ft/ft
Shear Checks
Required capacity = 571.754 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.625 in
Shear Reinforcement Area = 3.682 sq in/ft run
Shear Capacity Provided = 860.970 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 317.422 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.875	2.500
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 537.778 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

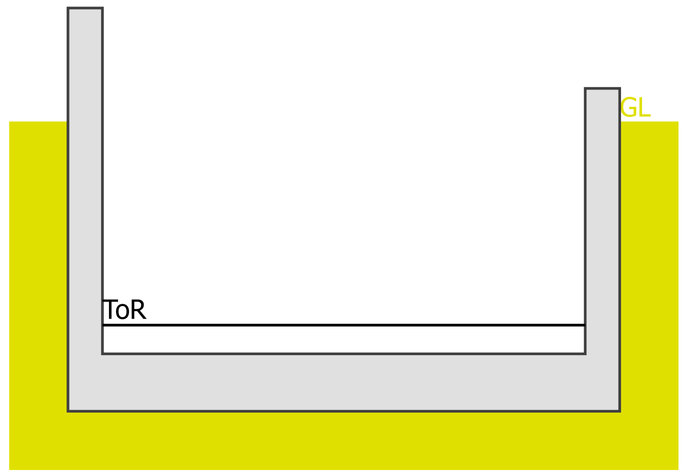
Fresno Grade Separation
Preliminary Design

Section at 10904+ 0.000
Original Ground Level 289.710
Groundwater Level 234.050
Top of Rail 272.092
Top of Base 269.592
Founding Level 264.592

UN-BRACED U-TROUGH
Trough Depth = 20.118 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 20.118 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 551.601 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.75	0.00	0.00
Smaller Bar	1.75	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.500	2.625
Area (sq-in)	4.811	0.000

Neutral Axis Depth = 7.074 in
Section is Tension controlled.
Reinforcement Strain 1.036e-02
Compression-block depth = 5.659 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 620.640 kip-ft/ft
Shear Checks
Required capacity = 573.147 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.625 in
Shear Reinforcement Area = 3.682 sq in/ft run
Shear Capacity Provided = 859.285 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 317.422 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.875	2.500
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 537.778 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip

SECTION AT 20.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 547.499 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.75	0.00	0.00
Smaller Bar	1.75	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.625	2.500
Area (sq-in)	4.811	0.000

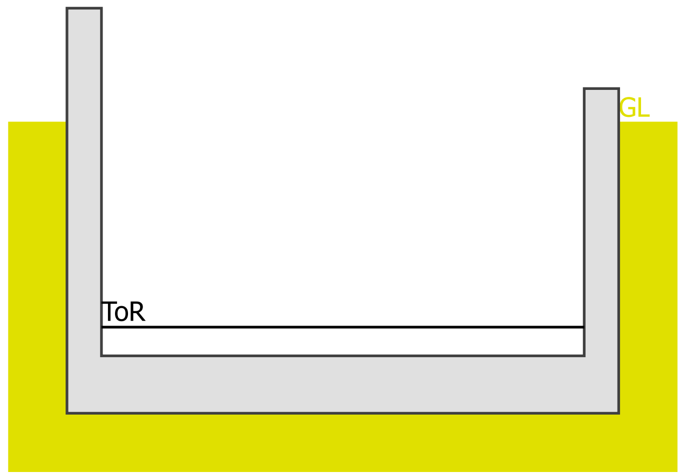
Neutral Axis Depth = 7.074 in
Section is Tension controlled.
Reinforcement Strain 1.041e-02
Compression-block depth = 5.659 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 623.346 kip-ft/ft
Shear Checks
Required capacity = 537.778 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 564.669 kip

Section at 10904+50.000
Original Ground Level 288.920
Groundwater Level 234.025
Top of Rail 271.142
Top of Base 268.642
Founding Level 263.642

UN-BRACED U-TROUGH
Trough Depth = 20.278 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 20.278 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 557.208 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.75	0.00	0.00
Smaller Bar	1.75	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.500	2.625
Area (sq-in)	4.811	0.000

Neutral Axis Depth = 7.074 in
Section is Tension controlled.
Reinforcement Strain 1.036e-02
Compression-block depth = 5.659 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 620.640 kip-ft/ft
Shear Checks
Required capacity = 573.849 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.625 in
Shear Reinforcement Area = 3.682 sq in/ft run
Shear Capacity Provided = 859.285 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 317.422 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.875	2.500
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 537.778 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip

SECTION AT 20.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 547.499 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.75	0.00	0.00
Smaller Bar	1.75	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.625	2.500
Area (sq-in)	4.811	0.000

Neutral Axis Depth = 7.074 in
Section is Tension controlled.
Reinforcement Strain 1.041e-02
Compression-block depth = 5.659 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 623.346 kip-ft/ft
Shear Checks
Required capacity = 537.778 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 564.669 kip

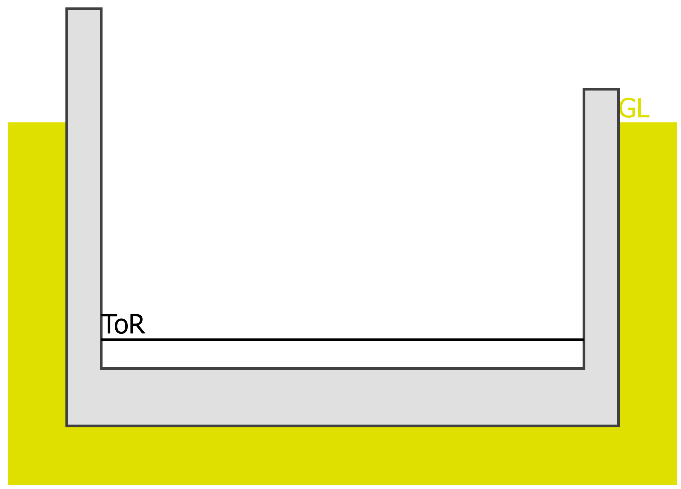
California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10905+ 0.000
Original Ground Level 288.990
Groundwater Level 234.000
Top of Rail 270.192
Top of Base 267.692
Founding Level 262.692

UN-BRACED U-TROUGH
Trough Depth = 21.298 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 21.298 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 594.597 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.75	0.00	0.00
Smaller Bar	1.75	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.500	2.625
Area (sq-in)	4.811	0.000

Neutral Axis Depth = 7.074 in
Section is Tension controlled.
Reinforcement Strain 1.036e-02
Compression-block depth = 5.659 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 620.640 kip-ft/ft
Shear Checks
Required capacity = 578.440 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.625 in
Shear Reinforcement Area = 3.682 sq in/ft run
Shear Capacity Provided = 859.285 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 317.422 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.875	2.500
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 537.778 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip

SECTION AT 20.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 547.499 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.75	0.00	0.00
Smaller Bar	1.75	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.625	2.500
Area (sq-in)	4.811	0.000

Neutral Axis Depth = 7.074 in
Section is Tension controlled.
Reinforcement Strain 1.041e-02
Compression-block depth = 5.659 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 623.346 kip-ft/ft
Shear Checks
Required capacity = 537.778 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 564.669 kip

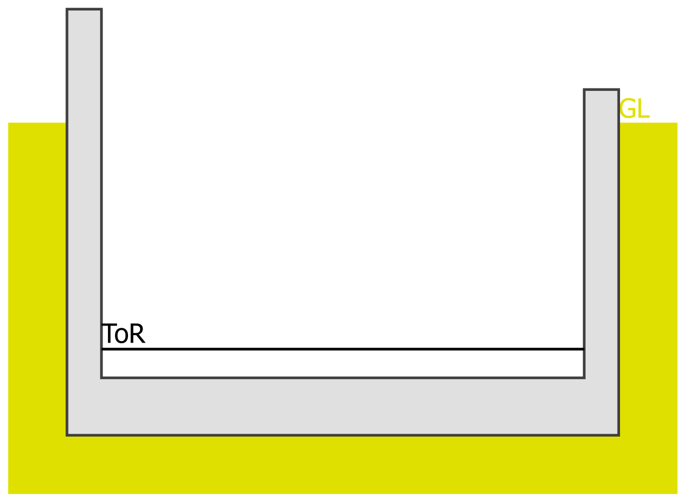
California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10905+50.000
Original Ground Level 288.850
Groundwater Level 237.833
Top of Rail 269.243
Top of Base 266.743
Founding Level 261.743

UN-BRACED U-TROUGH
Trough Depth = 22.107 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 22.107 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 626.298 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.88	0.00	0.00
Smaller Bar	1.88	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.438	2.625
Area (sq-in)	5.522	0.000

Neutral Axis Depth = 8.121 in
Section is Tension controlled.
Reinforcement Strain 8.613e-03
Compression-block depth = 6.497 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 700.512 kip-ft/ft
Shear Checks
Required capacity = 582.216 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.625 in
Shear Reinforcement Area = 3.682 sq in/ft run
Shear Capacity Provided = 859.285 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 317.422 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.875	2.500
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 537.778 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip

SECTION AT 20.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 547.499 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.75	0.00	0.00
Smaller Bar	1.75	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.625	2.500
Area (sq-in)	4.811	0.000

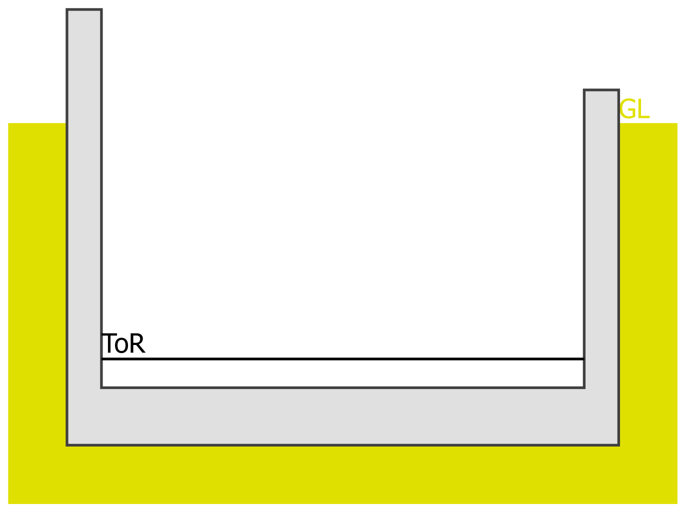
Neutral Axis Depth = 7.074 in
Section is Tension controlled.
Reinforcement Strain 1.041e-02
Compression-block depth = 5.659 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 623.346 kip-ft/ft
Shear Checks
Required capacity = 537.778 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 564.669 kip

Section at 10906+ 0.000
Original Ground Level 288.700
Groundwater Level 241.667
Top of Rail 268.293
Top of Base 265.793
Founding Level 260.793

UN-BRACED U-TROUGH
Trough Depth = 22.907 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 22.907 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 659.404 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.88	0.00	0.00
Smaller Bar	1.88	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.438	2.625
Area (sq-in)	5.522	0.000

Neutral Axis Depth = 8.121 in
Section is Tension controlled.
Reinforcement Strain 8.613e-03
Compression-block depth = 6.497 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 700.512 kip-ft/ft
Shear Checks
Required capacity = 586.057 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.625 in
Shear Reinforcement Area = 3.682 sq in/ft run
Shear Capacity Provided = 857.600 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 317.422 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.875	2.500
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 537.778 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip

SECTION AT 20.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 547.499 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.75	0.00	0.00
Smaller Bar	1.75	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.625	2.500
Area (sq-in)	4.811	0.000

Neutral Axis Depth = 7.074 in
Section is Tension controlled.
Reinforcement Strain 1.041e-02
Compression-block depth = 5.659 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 623.346 kip-ft/ft
Shear Checks
Required capacity = 537.778 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 564.669 kip

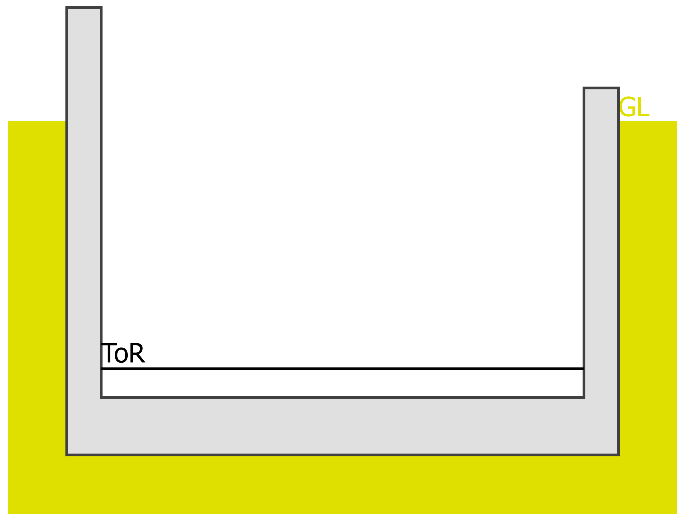
California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10906+50.000
Original Ground Level 288.770
Groundwater Level 245.500
Top of Rail 267.343
Top of Base 264.843
Founding Level 259.843

UN-BRACED U-TROUGH
Trough Depth = 23.927 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 23.927 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 704.279 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 2.00 0.00 0.00
Smaller Bar 2.00 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 31.375 2.625
Area (sq-in) 6.283 0.000
Neutral Axis Depth = 9.240 in
Section is Tension controlled.
Reinforcement Strain 7.187e-03
Compression-block depth = 7.392 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 782.606 kip-ft/ft
Shear Checks
Required capacity = 591.119 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.625 in
Shear Reinforcement Area = 3.682 sq in/ft run
Shear Capacity Provided = 857.600 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 317.422 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.25 0.00 0.00
Smaller Bar 1.25 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 31.875 2.500
Area (sq-in) 2.454 0.000
Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 537.778 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip

SECTION AT 20.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 547.499 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.75 0.00 0.00
Smaller Bar 1.75 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 31.625 2.500
Area (sq-in) 4.811 0.000
Neutral Axis Depth = 7.074 in
Section is Tension controlled.
Reinforcement Strain 1.041e-02
Compression-block depth = 5.659 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 623.346 kip-ft/ft
Shear Checks
Required capacity = 537.778 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 564.669 kip

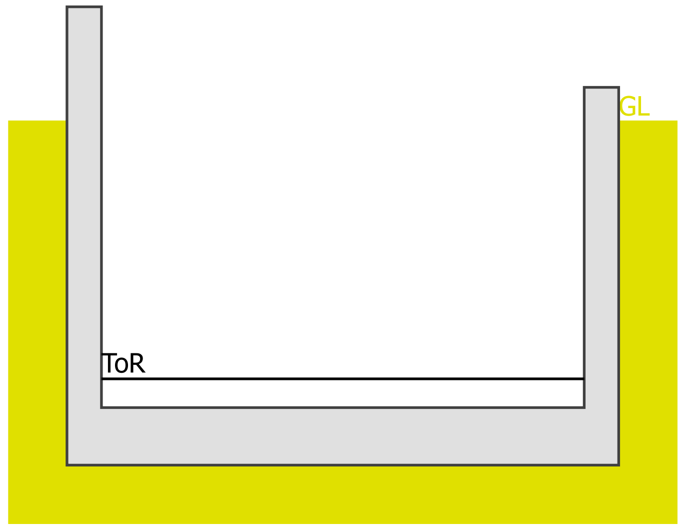
California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10907+ 0.000
Original Ground Level 288.730
Groundwater Level 249.333
Top of Rail 266.393
Top of Base 263.893
Founding Level 258.893

UN-BRACED U-TROUGH
Trough Depth = 24.837 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 24.837 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 746.902 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	2.00	0.00	0.00
Smaller Bar	2.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.375	2.625
Area (sq-in)	6.283	0.000

Neutral Axis Depth = 9.240 in
Section is Tension controlled.
Reinforcement Strain 7.187e-03
Compression-block depth = 7.392 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 782.606 kip-ft/ft
Shear Checks
Required capacity = 595.789 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.625 in
Shear Reinforcement Area = 3.682 sq in/ft run
Shear Capacity Provided = 855.915 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 317.422 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.875	2.500
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 537.778 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip

SECTION AT 20.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 547.499 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.75	0.00	0.00
Smaller Bar	1.75	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.625	2.500
Area (sq-in)	4.811	0.000

Neutral Axis Depth = 7.074 in
Section is Tension controlled.
Reinforcement Strain 1.041e-02
Compression-block depth = 5.659 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 623.346 kip-ft/ft
Shear Checks
Required capacity = 537.778 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 564.669 kip

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

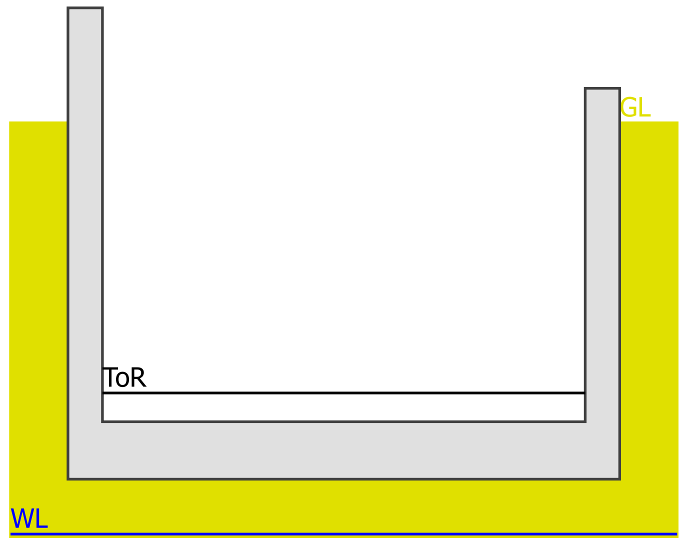
Fresno Grade Separation
Preliminary Design

Section at 10907+50.000
Original Ground Level 288.950
Groundwater Level 253.167
Top of Rail 265.444
Top of Base 262.944
Founding Level 257.944

UN-BRACED U-TROUGH
Trough Depth = 26.006 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 26.006 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 805.402 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	2.00	0.50	0.00
Smaller Bar	2.00	0.50	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.184	2.625
Area (sq-in)	6.676	0.000

Neutral Axis Depth = 9.817 in
Section is Tension controlled.
Reinforcement Strain 6.587e-03
Compression-block depth = 7.854 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 818.836 kip-ft/ft
Shear Checks
Required capacity = 602.007 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.625 in
Shear Reinforcement Area = 3.682 sq in/ft run
Shear Capacity Provided = 850.762 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 317.422 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.875	2.500
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 537.778 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip

SECTION AT 20.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 547.499 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.75	0.00	0.00
Smaller Bar	1.75	0.00	0.00

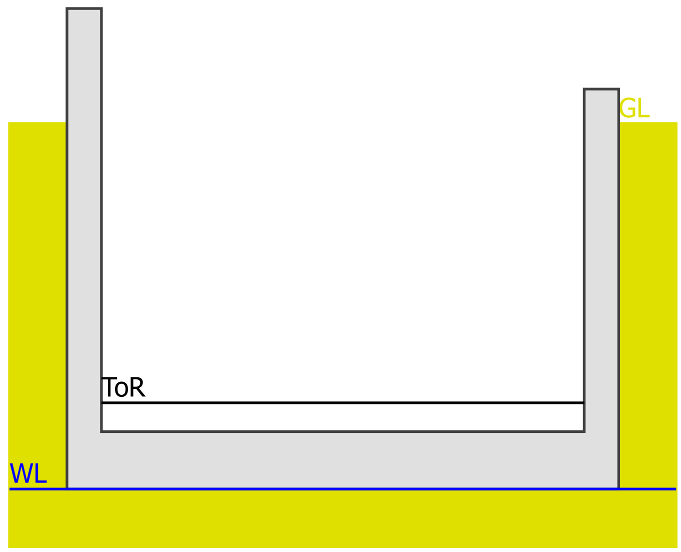
Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.625	2.500
Area (sq-in)	4.811	0.000

Neutral Axis Depth = 7.074 in
Section is Tension controlled.
Reinforcement Strain 1.041e-02
Compression-block depth = 5.659 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 623.346 kip-ft/ft
Shear Checks
Required capacity = 537.778 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 564.669 kip

Section at 10908+ 0.000
Original Ground Level 288.770
Groundwater Level 257.000
Top of Rail 264.494
Top of Base 261.994
Founding Level 256.994

UN-BRACED U-TROUGH
Trough Depth = 26.776 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 420.000 psf
FoS Against Flotation = 3675.860



WALL ROOT SECTION at 26.776 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 846.229 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	2.00	0.75	0.00
Smaller Bar	2.00	0.75	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	30.959	2.625
Area (sq-in)	7.167	0.000

Neutral Axis Depth = 10.539 in
Section is Tension controlled.
Reinforcement Strain 5.931e-03
Compression-block depth = 8.431 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 862.478 kip-ft/ft
Shear Checks
Required capacity = 606.230 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.625 in
Shear Reinforcement Area = 3.682 sq in/ft run
Shear Capacity Provided = 844.698 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 317.422 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.875	2.500
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 537.778 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip

SECTION AT 20.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 547.499 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.75	0.00	0.00
Smaller Bar	1.75	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.625	2.500
Area (sq-in)	4.811	0.000

Neutral Axis Depth = 7.074 in
Section is Tension controlled.
Reinforcement Strain 1.041e-02
Compression-block depth = 5.659 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 623.346 kip-ft/ft
Shear Checks
Required capacity = 537.778 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 564.669 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

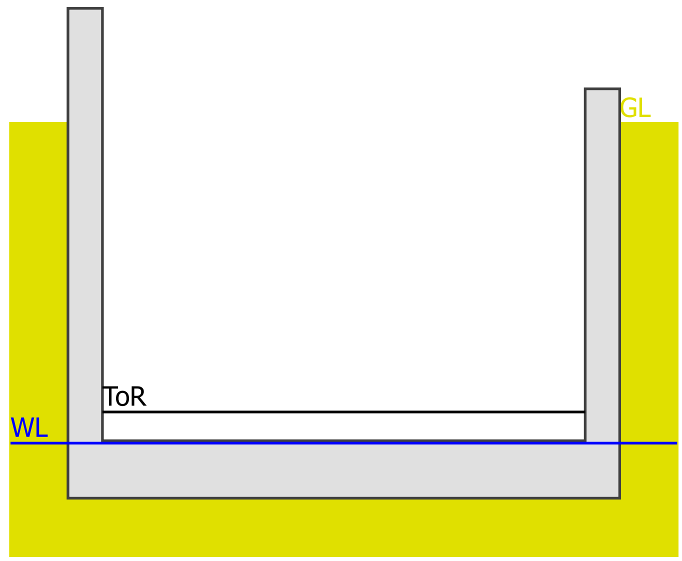
Fresno Grade Separation
Preliminary Design

Section at 10908+50.000
Original Ground Level 288.660
Groundwater Level 260.833
Top of Rail 263.544
Top of Base 261.044
Founding Level 256.044

UN-BRACED U-TROUGH
Trough Depth = 27.616 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = 4.836



WALL ROOT SECTION at 27.616 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 892.935 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	2.00	1.00	0.00
Smaller Bar	2.00	1.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	30.675	2.625
Area (sq-in)	7.854	0.000

Neutral Axis Depth = 11.550 in
Section is Tension controlled.
Reinforcement Strain 5.149e-03
Compression-block depth = 9.240 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 920.860 kip-ft/ft
Shear Checks
Required capacity = 610.955 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.625 in
Shear Reinforcement Area = 3.682 sq in/ft run
Shear Capacity Provided = 837.045 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 317.422 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.875	2.500
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 537.778 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip

SECTION AT 20.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 547.499 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.75	0.00	0.00
Smaller Bar	1.75	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.625	2.500
Area (sq-in)	4.811	0.000

Neutral Axis Depth = 7.074 in
Section is Tension controlled.
Reinforcement Strain 1.041e-02
Compression-block depth = 5.659 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 623.346 kip-ft/ft
Shear Checks
Required capacity = 537.778 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 564.669 kip

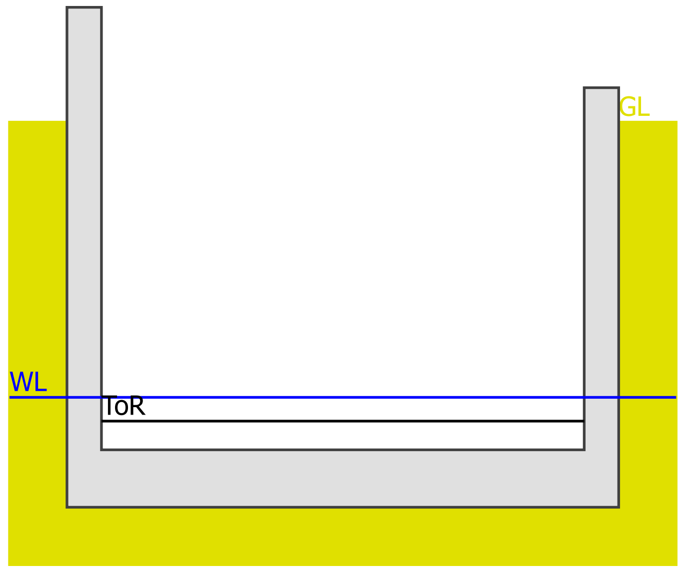
California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10909+ 0.000
Original Ground Level 288.630
Groundwater Level 264.667
Top of Rail 262.597
Top of Base 260.097
Founding Level 255.097

UN-BRACED U-TROUGH
Trough Depth = 28.533 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 420.000 psf
FoS Against Flotation = 2.450



WALL ROOT SECTION at 28.533 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 936.734 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	2.00	1.25	0.00
Smaller Bar	2.00	1.25	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	30.357	2.625
Area (sq-in)	8.738	0.000

Neutral Axis Depth = 12.849 in
Section is in transition to Compression Control.
Reinforcement Strain 4.325e-03
Compression-block depth = 10.279 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 991.507 kip-ft/ft
Shear Checks
Required capacity = 576.459 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.625 in
Shear Reinforcement Area = 3.682 sq in/ft run
Shear Capacity Provided = 828.465 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 317.422 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.875	2.500
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 537.778 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip

SECTION AT 20.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 547.499 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.75	0.00	0.00
Smaller Bar	1.75	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.625	2.500
Area (sq-in)	4.811	0.000

Neutral Axis Depth = 7.074 in
Section is Tension controlled.
Reinforcement Strain 1.041e-02
Compression-block depth = 5.659 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 623.346 kip-ft/ft
Shear Checks
Required capacity = 537.778 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 564.669 kip

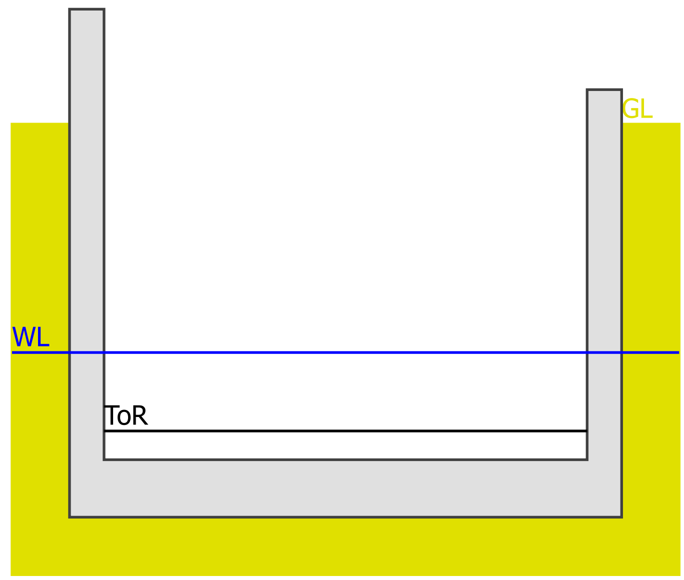
California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10909+50.000
Original Ground Level 288.370
Groundwater Level 268.500
Top of Rail 261.673
Top of Base 259.173
Founding Level 254.173

UN-BRACED U-TROUGH
Trough Depth = 29.197 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 420.000 psf
FoS Against Flotation = 1.651



WALL ROOT SECTION at 29.197 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 958.982 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	2.00	1.25	0.00
Smaller Bar	2.00	1.25	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	30.357	2.625
Area (sq-in)	8.738	0.000

Neutral Axis Depth = 12.849 in
Section is in transition to Compression Control.
Reinforcement Strain 4.325e-03
Compression-block depth = 10.279 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 991.507 kip-ft/ft
Shear Checks
Required capacity = 577.755 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.625 in
Shear Reinforcement Area = 3.682 sq in/ft run
Shear Capacity Provided = 828.465 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 317.422 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.875	2.500
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 537.778 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip

SECTION AT 20.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 547.492 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.75	0.00	0.00
Smaller Bar	1.75	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.625	2.500
Area (sq-in)	4.811	0.000

Neutral Axis Depth = 7.074 in
Section is Tension controlled.
Reinforcement Strain 1.041e-02
Compression-block depth = 5.659 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 623.346 kip-ft/ft
Shear Checks
Required capacity = 537.778 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 564.669 kip

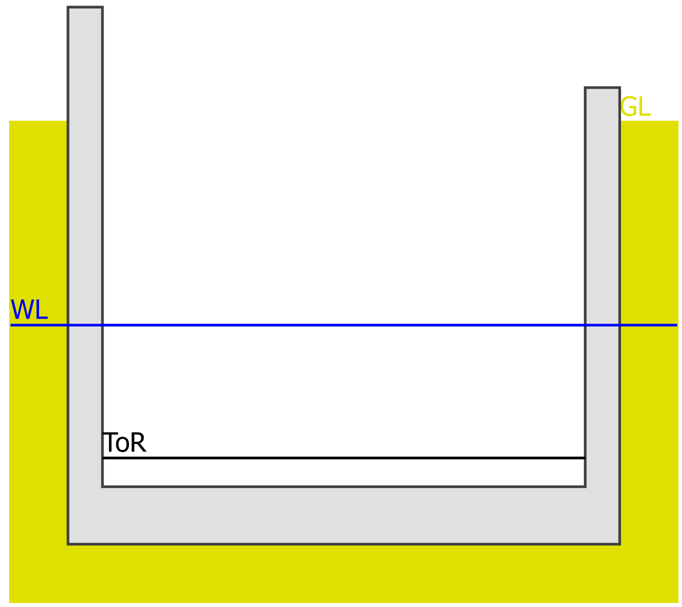
Fresno Grade Separation
Preliminary Design

Section at 10910+ 0.000
Original Ground Level 290.000
Groundwater Level 272.333
Top of Rail 260.775
Top of Base 258.275
Founding Level 253.275

UN-BRACED U-TROUGH
Trough Depth = 31.725 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = 1.283
Additional tie down force required 12.406 kips



WALL ROOT SECTION at 31.725 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 1112.426 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	2.00	1.75	0.00
Smaller Bar	2.00	1.75	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	29.695	2.625
Area (sq-in)	11.094	0.000

Neutral Axis Depth = 16.314 in
Section is in transition to Compression Control.
Reinforcement Strain 2.769e-03
Compression-block depth = 13.051 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 1156.638 kip-ft/ft
Shear Checks
Required capacity = 587.873 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.625 in
Shear Reinforcement Area = 3.682 sq in/ft run
Shear Capacity Provided = 810.618 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 317.422 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.875	2.500
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 537.778 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip

SECTION AT 20.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 545.543 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

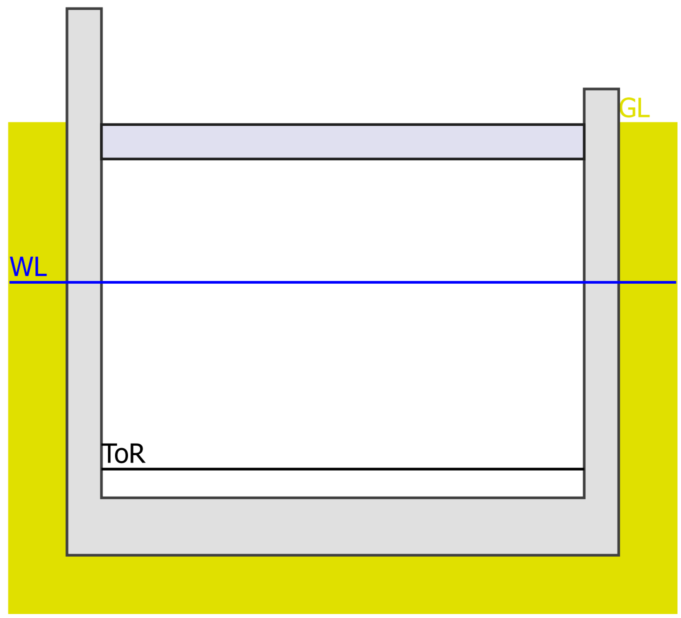
Layer	T1	T2	C1
Larger Bar	1.62	0.00	0.00
Smaller Bar	1.62	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.688	2.500
Area (sq-in)	4.148	0.000

Neutral Axis Depth = 6.100 in
Section is Tension controlled.
Reinforcement Strain 1.258e-02
Compression-block depth = 4.880 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 545.920 kip-ft/ft
Shear Checks
Required capacity = 537.778 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 564.669 kip

Section at 10910+50.000
Original Ground Level 290.000
Groundwater Level 276.167
Top of Rail 259.903
Top of Base 257.403
Founding Level 252.403



BRACED U-TROUGH
Trough Depth = 32.597 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 420.000 psf
FoS Against Flotation = 1.040
Additional tie down force required 32.729 kips

BASE OF WALL AT 32.597 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 122.177 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 26.094 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL AT 19.313 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 138.711 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL PROP
Prop Spacing = 20.000 ft
Prop Force = 61.747 kip/ft
Force per prop = 1234.932 kips

WALL AT PROP LEVEL (1.597 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 260.731 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.188	2.250
Area (sq-in)	1.988	0.000

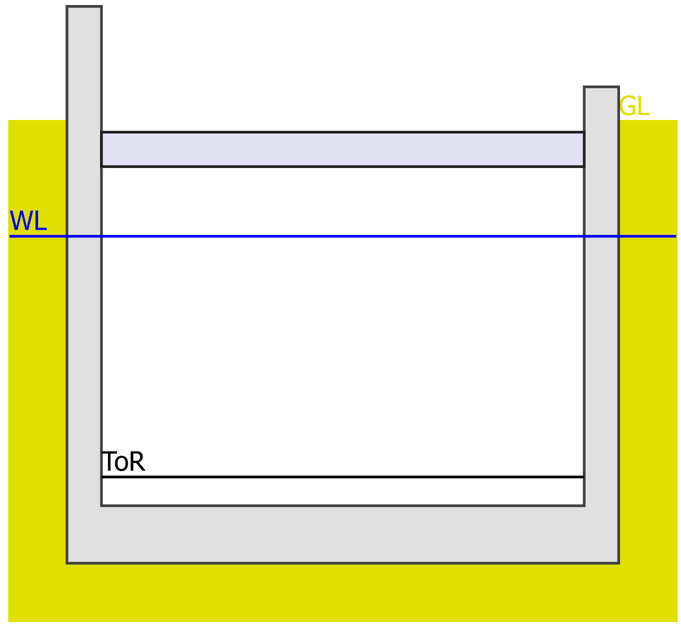
Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 19.311 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.209 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10911+ 0.000
Original Ground Level 290.000
Groundwater Level 280.000
Top of Rail 259.058
Top of Base 256.558
Founding Level 251.558



BRACED U-TROUGH
Trough Depth = 33.442 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = 0.878
Additional tie down force required 52.960 kips

BASE OF WALL AT 33.442 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 174.074 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 35.259 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL AT 19.231 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 137.388 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL PROP
Prop Spacing = 20.000 ft
Prop Force = 55.136 kip/ft
Force per prop = 1102.715 kips

WALL AT PROP LEVEL (2.442 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 261.385 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

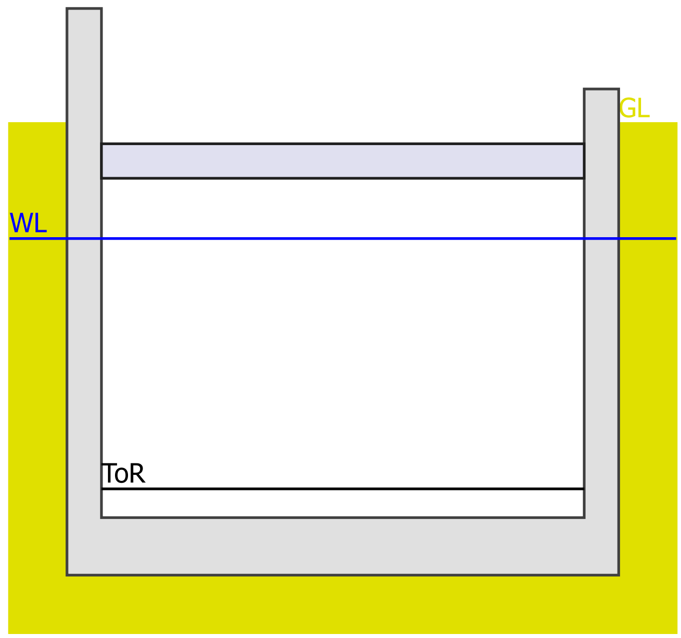
Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.188	2.250
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 21.882 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.209 kip

Section at 10911+50.000
Original Ground Level 290.000
Groundwater Level 280.000
Top of Rail 258.238
Top of Base 255.738
Founding Level 250.738



BRACED U-TROUGH
Trough Depth = 34.262 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = 0.862
Additional tie down force required 55.875 kips

BASE OF WALL AT 34.262 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 183.827 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 36.935 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL AT 19.118 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 135.582 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL PROP
Prop Spacing = 20.000 ft
Prop Force = 50.457 kip/ft
Force per prop = 1009.141 kips

WALL AT PROP LEVEL (3.262 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 262.279 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

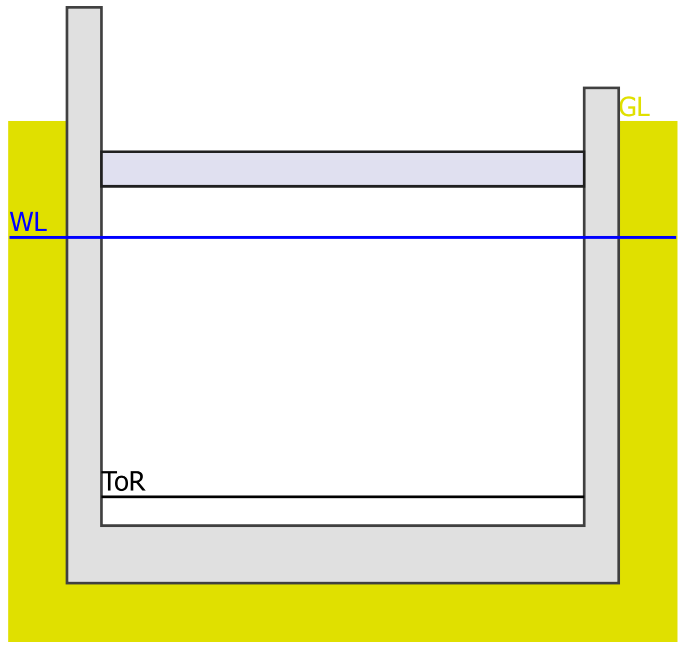
Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.188	2.250
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 22.779 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.209 kip

Section at 10912+ 0.000
Original Ground Level 290.000
Groundwater Level 280.000
Top of Rail 257.445
Top of Base 254.945
Founding Level 249.945



BRACED U-TROUGH
Trough Depth = 35.055 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = 0.848
Additional tie down force required 58.697 kips

BASE OF WALL AT 35.055 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 193.384 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 38.569 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL AT 18.977 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 133.345 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL PROP
Prop Spacing = 20.000 ft
Prop Force = 47.005 kip/ft
Force per prop = 940.105 kips

WALL AT PROP LEVEL (4.055 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 263.389 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.188	2.250
Area (sq-in)	1.988	0.000

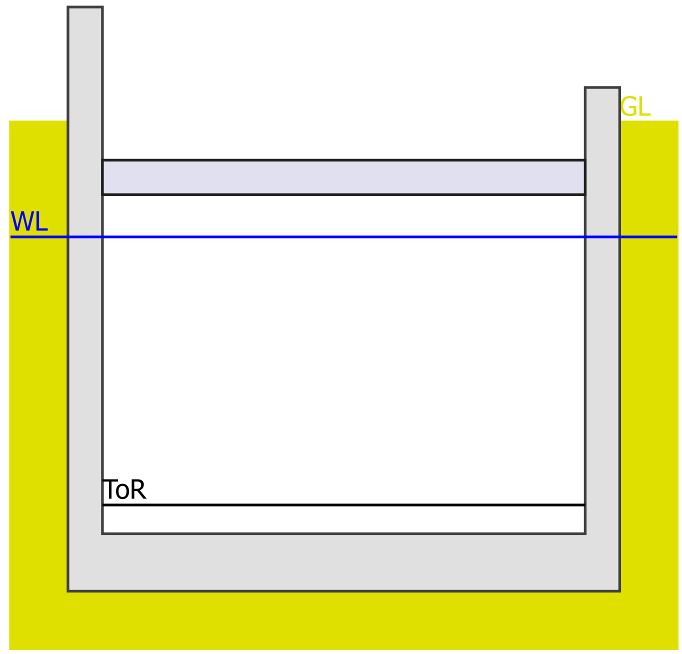
Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 23.709 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.209 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10912+50.000
Original Ground Level 290.000
Groundwater Level 280.000
Top of Rail 256.677
Top of Base 254.177
Founding Level 249.177



BRACED U-TROUGH
Trough Depth = 35.823 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = 0.835
Additional tie down force required 61.426 kips

BASE OF WALL AT 35.823 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 202.710 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 40.159 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL AT 18.812 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 130.730 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL PROP
Prop Spacing = 20.000 ft
Prop Force = 44.381 kip/ft
Force per prop = 887.612 kips

WALL AT PROP LEVEL (4.823 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 264.692 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

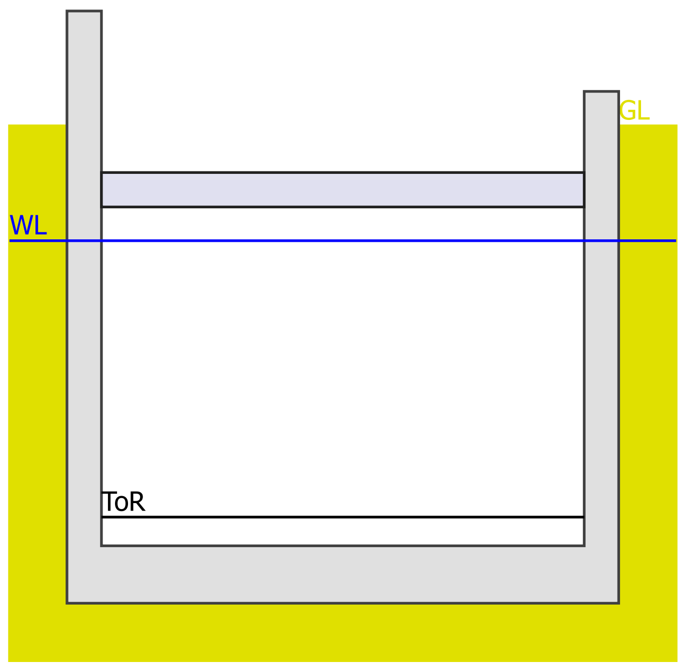
Layer	Tension	Compression
Depth (in)	32.188	2.250
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 24.669 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.209 kip

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10913+ 0.000
Original Ground Level 290.000
Groundwater Level 280.000
Top of Rail 255.936
Top of Base 253.436
Founding Level 248.436



BRACED U-TROUGH
Trough Depth = 36.564 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = 0.822
Additional tie down force required 64.062 kips

BASE OF WALL AT 36.564 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 211.774 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 41.699 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL AT 18.626 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 127.787 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL PROP
Prop Spacing = 20.000 ft
Prop Force = 42.340 kip/ft
Force per prop = 846.794 kips

WALL AT PROP LEVEL (5.564 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 266.164 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.188	2.250
Area (sq-in)	1.988	0.000

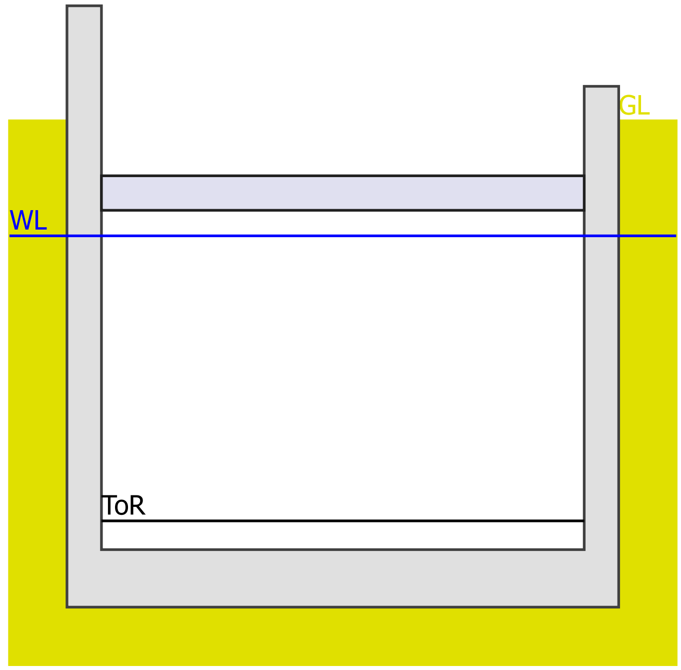
Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 25.656 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.209 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10913+50.000
Original Ground Level 290.000
Groundwater Level 280.000
Top of Rail 255.221
Top of Base 252.721
Founding Level 247.721



BRACED U-TROUGH
Trough Depth = 37.279 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 420.000 psf
FoS Against Flotation = 0.811
Additional tie down force required 66.605 kips

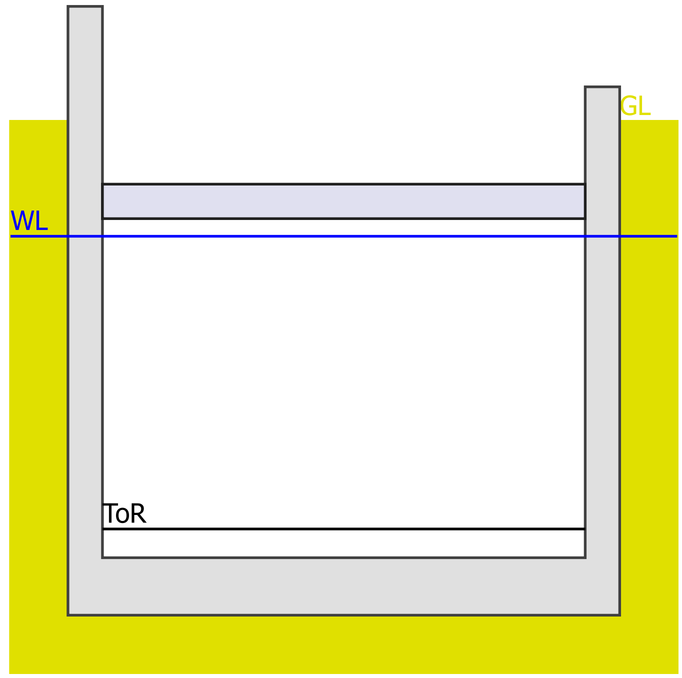
BASE OF WALL AT 37.279 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 220.550 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.00 0.00 0.00
Smaller Bar 1.00 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.250 2.250
Area (sq-in) 1.571 0.000
Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 43.189 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL AT 18.421 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 124.565 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.00 0.00 0.00
Smaller Bar 1.00 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.250 2.250
Area (sq-in) 1.571 0.000
Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL PROP
Prop Spacing = 20.000 ft
Prop Force = 40.726 kip/ft
Force per prop = 814.510 kips

WALL AT PROP LEVEL (6.279 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 267.783 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.12 0.00 0.00
Smaller Bar 1.12 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.188 2.250
Area (sq-in) 1.988 0.000
Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 26.665 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.209 kip

Section at 10914+ 0.000
Original Ground Level 290.000
Groundwater Level 280.000
Top of Rail 254.532
Top of Base 252.032
Founding Level 247.032



BRACED U-TROUGH
Trough Depth = 37.968 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 420.000 psf
FoS Against Flotation = 0.801
Additional tie down force required 69.056 kips

BASE OF WALL AT 37.968 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 229.016 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.12 0.00 0.00
Smaller Bar 1.12 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.188 2.250
Area (sq-in) 1.988 0.000
Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 44.625 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.209 kip

WALL AT 18.200 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 121.112 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.00 0.00 0.00
Smaller Bar 1.00 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.250 2.250
Area (sq-in) 1.571 0.000
Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

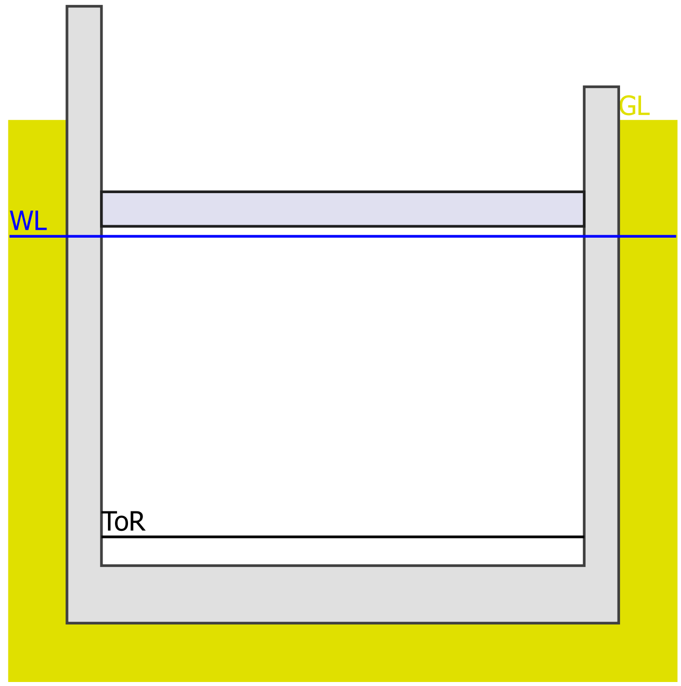
WALL PROP
Prop Spacing = 20.000 ft
Prop Force = 39.432 kip/ft
Force per prop = 788.644 kips

WALL AT PROP LEVEL (6.968 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 269.528 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.12 0.00 0.00
Smaller Bar 1.12 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.188 2.250
Area (sq-in) 1.988 0.000
Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 27.691 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.209 kip

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10914+50.000
Original Ground Level 290.000
Groundwater Level 280.000
Top of Rail 253.870
Top of Base 251.370
Founding Level 246.370



BRACED U-TROUGH
Trough Depth = 38.630 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = 0.791
Additional tie down force required 71.413 kips

BASE OF WALL AT 38.630 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 237.152 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.188	2.250
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 46.006 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.209 kip

WALL AT 17.966 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 117.470 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL PROP
Prop Spacing = 20.000 ft
Prop Force = 38.386 kip/ft
Force per prop = 767.716 kips

WALL AT PROP LEVEL (7.630 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 271.379 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.188	2.250
Area (sq-in)	1.988	0.000

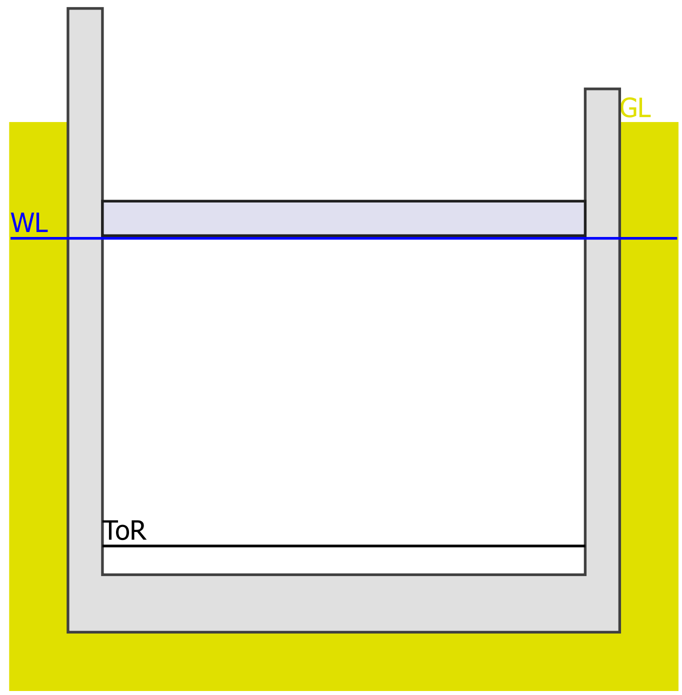
Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 28.731 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.209 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10915+ 0.000
Original Ground Level 290.000
Groundwater Level 280.000
Top of Rail 253.233
Top of Base 250.733
Founding Level 245.733



BRACED U-TROUGH
Trough Depth = 39.267 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 420.000 psf
FoS Against Flotation = 0.782
Additional tie down force required 73.677 kips

BASE OF WALL AT 39.267 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 244.945 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.188	2.250
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 47.330 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.209 kip

WALL AT 17.721 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 113.684 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL PROP
Prop Spacing = 20.000 ft
Prop Force = 37.533 kip/ft
Force per prop = 750.658 kips

WALL AT PROP LEVEL (8.267 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 273.314 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.188	2.250
Area (sq-in)	1.988	0.000

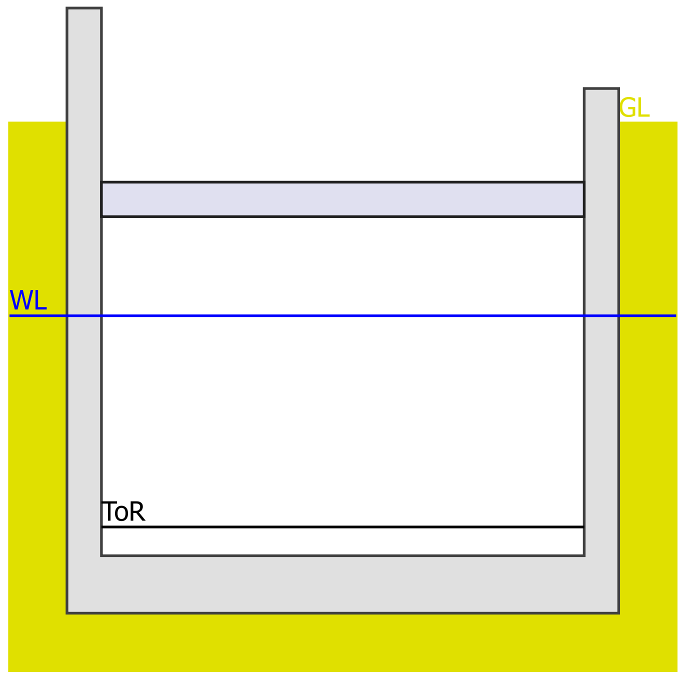
Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 29.780 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.209 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10915+50.000
Original Ground Level 287.770
Groundwater Level 271.000
Top of Rail 252.622
Top of Base 250.122
Founding Level 245.122



BRACED U-TROUGH
Trough Depth = 37.648 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 420.000 psf
FoS Against Flotation = 1.016
Additional tie down force required 37.501 kips

BASE OF WALL AT 37.648 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 140.397 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.00 0.00 0.00
Smaller Bar 1.00 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.250 2.250
Area (sq-in) 1.571 0.000
Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 29.719 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL AT 18.306 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 122.759 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.00 0.00 0.00
Smaller Bar 1.00 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.250 2.250
Area (sq-in) 1.571 0.000
Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL PROP
Prop Spacing = 20.000 ft
Prop Force = 40.005 kip/ft
Force per prop = 800.094 kips

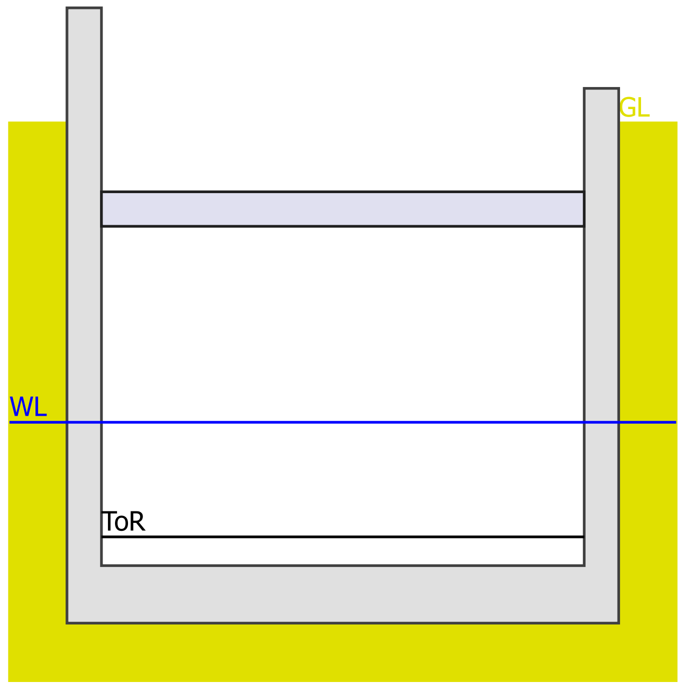
WALL AT PROP LEVEL (6.648 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 268.695 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.12 0.00 0.00
Smaller Bar 1.12 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.188 2.250
Area (sq-in) 1.988 0.000
Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 22.462 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.209 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10916+ 0.000
Original Ground Level 288.030
Groundwater Level 262.000
Top of Rail 252.038
Top of Base 249.538
Founding Level 244.538



BRACED U-TROUGH
Trough Depth = 38.492 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 420.000 psf
FoS Against Flotation = 1.521

BASE OF WALL AT 38.492 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 67.522 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.00 0.00 0.00
Smaller Bar 1.00 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.250 2.250
Area (sq-in) 1.571 0.000
Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 15.532 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL AT 18.017 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 118.257 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.00 0.00 0.00
Smaller Bar 1.00 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.250 2.250
Area (sq-in) 1.571 0.000
Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL PROP
Prop Spacing = 10.000 ft
Prop Force = 38.590 kip/ft
Force per prop = 385.903 kips

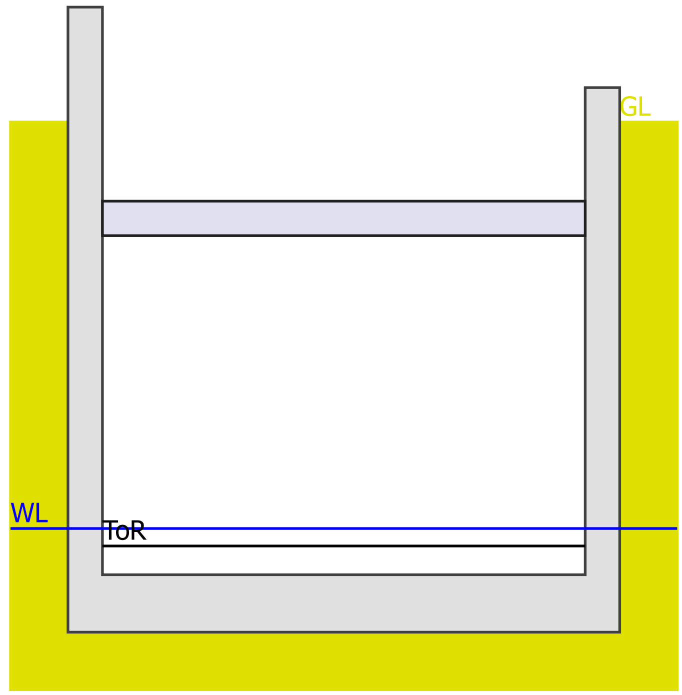
WALL AT PROP LEVEL (7.492 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 270.978 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.12 0.00 0.00
Smaller Bar 1.12 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.188 2.250
Area (sq-in) 1.988 0.000
Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 20.757 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.209 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10916+50.000
Original Ground Level 288.360
Groundwater Level 253.000
Top of Rail 251.480
Top of Base 248.980
Founding Level 243.980



BRACED U-TROUGH
Trough Depth = 39.380 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = 2.975

BASE OF WALL AT 39.380 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 89.759 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 7.691 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL AT 17.676 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 112.982 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL PROP
Prop Spacing = 10.000 ft
Prop Force = 37.395 kip/ft
Force per prop = 373.953 kips

WALL AT PROP LEVEL (8.380 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 273.674 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

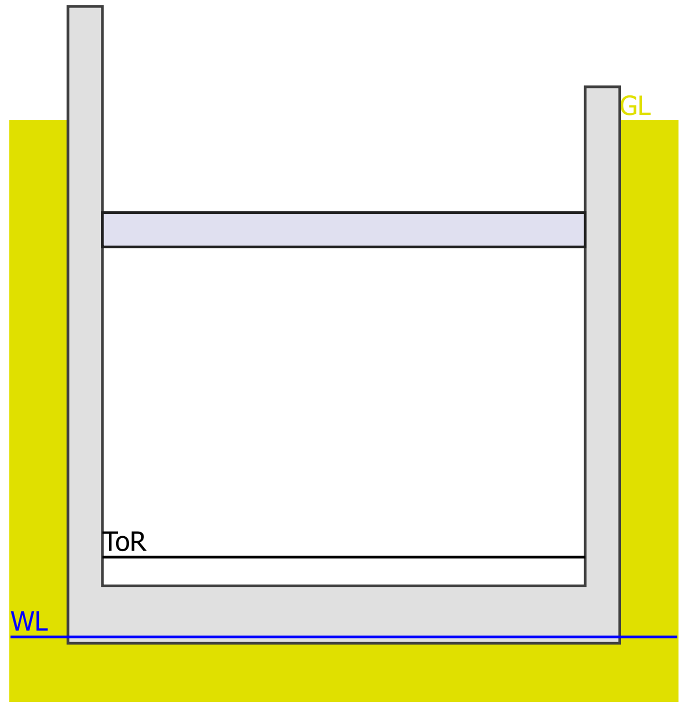
Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.188	2.250
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 20.898 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.209 kip

Section at 10917+ 0.000
Original Ground Level 288.890
Groundwater Level 244.000
Top of Rail 250.948
Top of Base 248.448
Founding Level 243.448



BRACED U-TROUGH
Trough Depth = 40.442 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 420.000 psf
FoS Against Flotation = 49.202

BASE OF WALL AT 40.442 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 94.585 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 7.723 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL AT 17.218 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 105.969 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL PROP
Prop Spacing = 10.000 ft
Prop Force = 36.277 kip/ft
Force per prop = 362.773 kips

WALL AT PROP LEVEL (9.442 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 277.294 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

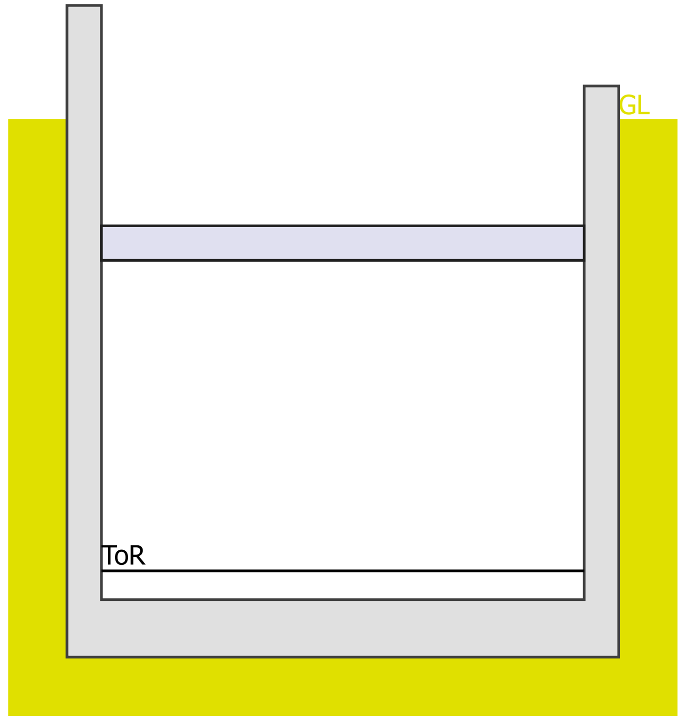
Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.188	2.250
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 21.481 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.209 kip

Fresno Grade Separation
Preliminary Design

Section at 10917+50.000
Original Ground Level 289.600
Groundwater Level 235.000
Top of Rail 250.442
Top of Base 247.942
Founding Level 242.942



BRACED U-TROUGH
Trough Depth = 41.658 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf

BASE OF WALL AT 41.658 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 150.960 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.00 0.00 0.00
Smaller Bar 1.00 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.250 2.250
Area (sq-in) 1.571 0.000
Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 15.581 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL AT 16.627 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 97.033 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.00 0.00 0.00
Smaller Bar 1.00 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.250 2.250
Area (sq-in) 1.571 0.000
Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL PROP
Prop Spacing = 10.000 ft
Prop Force = 35.324 kip/ft
Force per prop = 353.236 kips

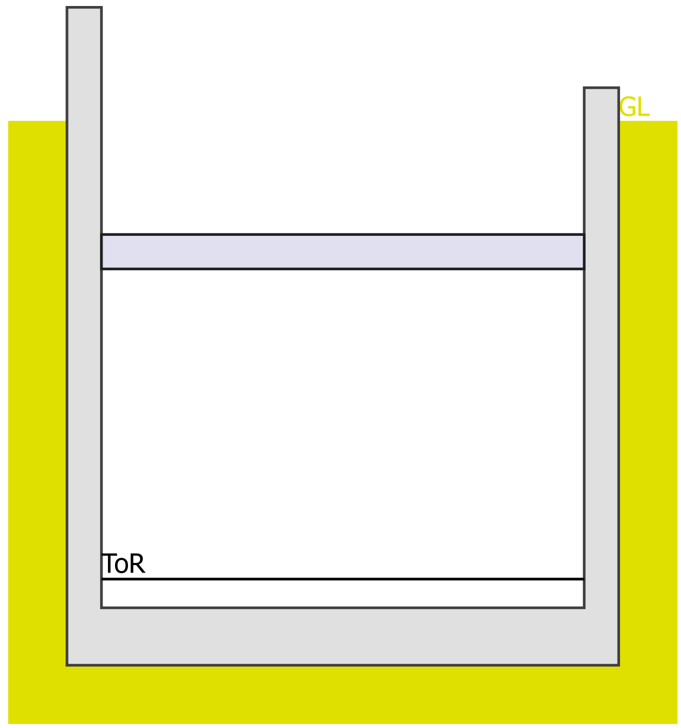
WALL AT PROP LEVEL (10.658 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 281.968 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.25 0.00 0.00
Smaller Bar 1.25 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.125 2.250
Area (sq-in) 2.454 0.000
Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 22.623 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10918+ 0.000
Original Ground Level 289.740
Groundwater Level 235.000
Top of Rail 249.962
Top of Base 247.462
Founding Level 242.462



BRACED U-TROUGH
Trough Depth = 42.278 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf

BASE OF WALL AT 42.278 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 145.764 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 14.817 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL AT 16.298 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 92.120 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL PROP
Prop Spacing = 10.000 ft
Prop Force = 34.946 kip/ft
Force per prop = 349.465 kips

WALL AT PROP LEVEL (11.278 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 284.568 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

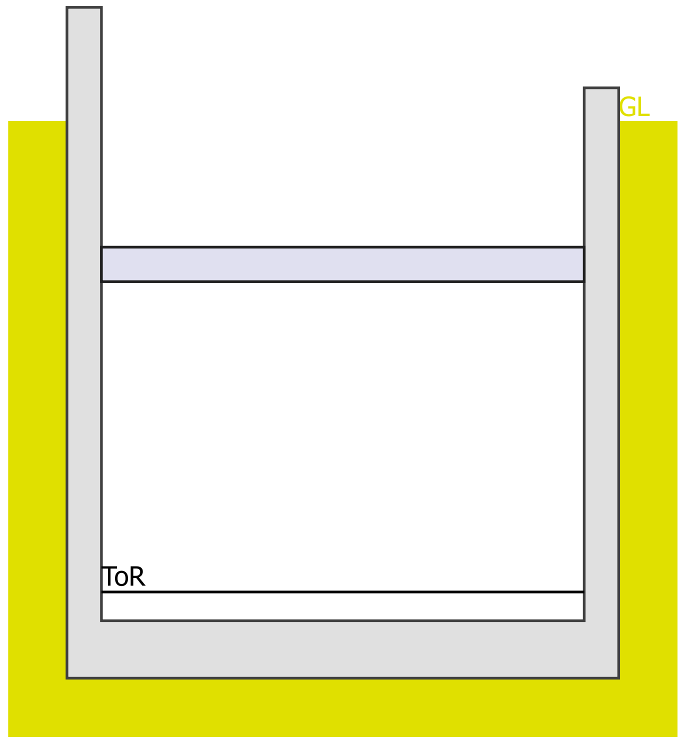
Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.125	2.250
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 22.921 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

Section at 10918+50.000
Original Ground Level 290.370
Groundwater Level 235.000
Top of Rail 249.508
Top of Base 247.008
Founding Level 242.008



BRACED U-TROUGH
Trough Depth = 43.362 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf

BASE OF WALL AT 43.362 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 142.586 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.00 0.00 0.00
Smaller Bar 1.00 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.250 2.250
Area (sq-in) 1.571 0.000
Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 13.993 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL AT 15.678 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 82.975 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.00 0.00 0.00
Smaller Bar 1.00 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.250 2.250
Area (sq-in) 1.571 0.000
Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL PROP
Prop Spacing = 10.000 ft
Prop Force = 34.434 kip/ft
Force per prop = 344.337 kips

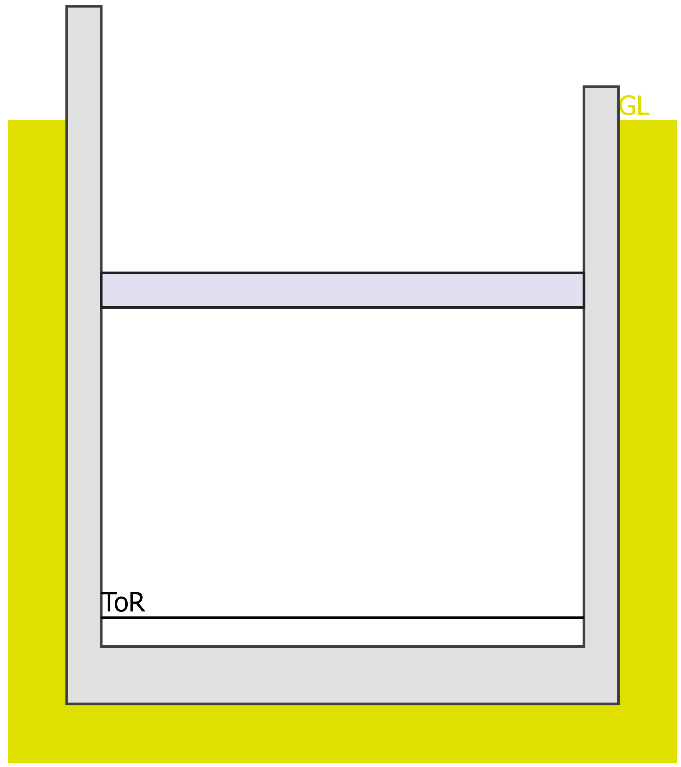
WALL AT PROP LEVEL (12.362 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 289.467 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.25 0.00 0.00
Smaller Bar 1.25 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.125 2.250
Area (sq-in) 2.454 0.000
Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 23.518 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10919+ 0.000
Original Ground Level 292.280
Groundwater Level 235.000
Top of Rail 249.081
Top of Base 246.581
Founding Level 241.581



BRACED U-TROUGH
Trough Depth = 45.699 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf

BASE OF WALL AT 45.699 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 143.765 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 12.871 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL AT 14.148 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 61.024 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL PROP
Prop Spacing = 10.000 ft
Prop Force = 33.834 kip/ft
Force per prop = 338.341 kips

WALL AT PROP LEVEL (14.699 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 301.564 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

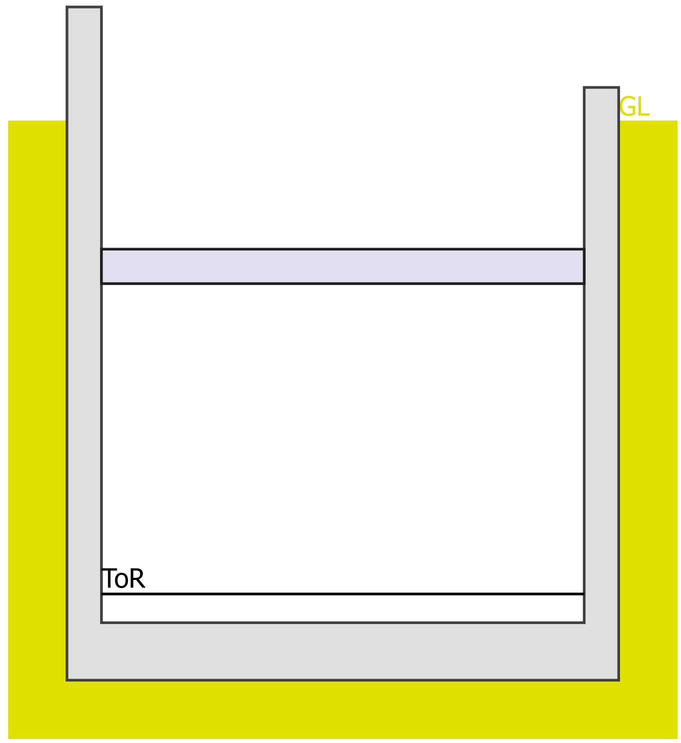
Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.125	2.250
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 24.951 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

Fresno Grade Separation
Preliminary Design

Section at 10919+50.000
Original Ground Level 289.750
Groundwater Level 235.000
Top of Rail 248.679
Top of Base 246.179
Founding Level 241.179



BRACED U-TROUGH
Trough Depth = 43.571 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf

BASE OF WALL AT 43.571 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 134.006 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 12.919 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL AT 15.552 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 81.134 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL PROP
Prop Spacing = 10.000 ft
Prop Force = 34.354 kip/ft
Force per prop = 343.540 kips

WALL AT PROP LEVEL (12.571 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 290.463 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.125	2.250
Area (sq-in)	2.454	0.000

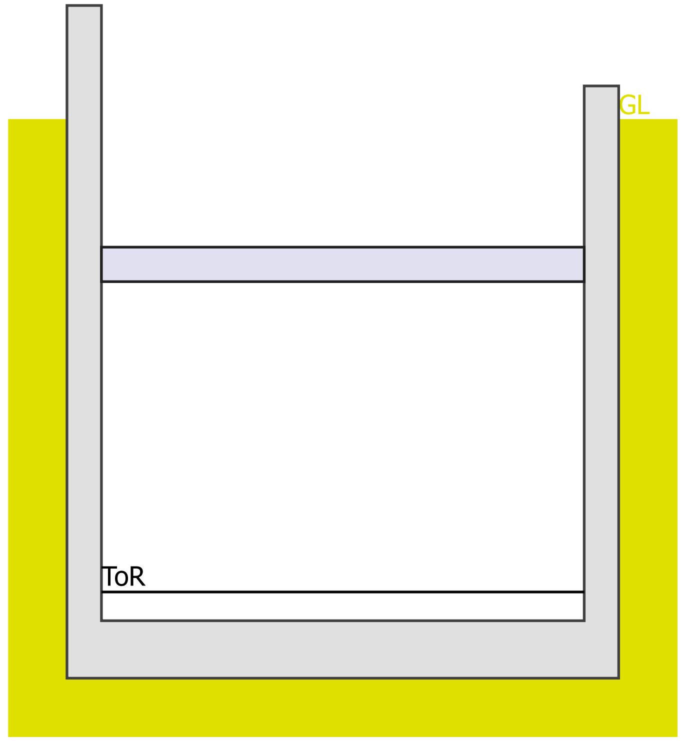
Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 23.561 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10920+ 0.000
Original Ground Level 289.330
Groundwater Level 235.000
Top of Rail 248.304
Top of Base 245.804
Founding Level 240.804



BRACED U-TROUGH
Trough Depth = 43.526 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf

BASE OF WALL AT 43.526 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 130.266 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 12.485 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL AT 15.579 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 81.530 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL PROP
Prop Spacing = 10.000 ft
Prop Force = 34.371 kip/ft
Force per prop = 343.706 kips

WALL AT PROP LEVEL (12.526 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 290.248 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.125	2.250
Area (sq-in)	2.454	0.000

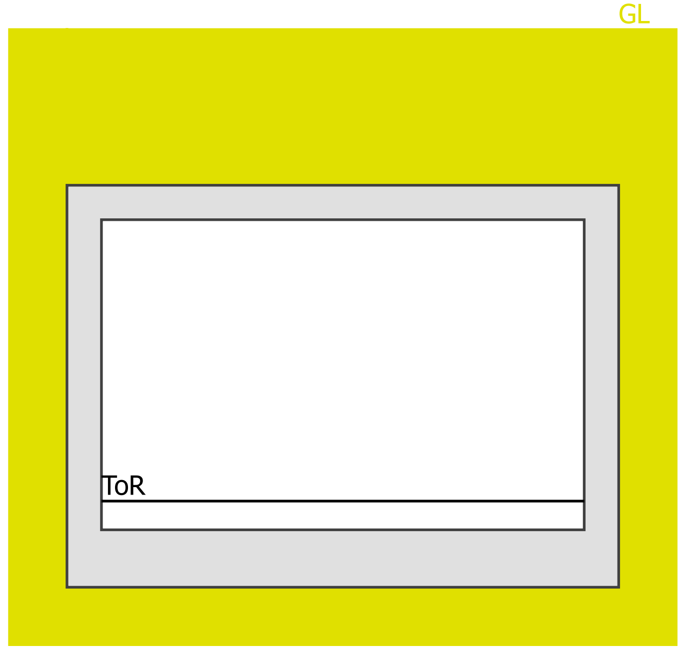
Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 23.500 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10920+50.000
Original Ground Level 289.010
Groundwater Level 235.000
Top of Rail 247.955
Top of Base 245.455
Founding Level 240.455



COVERED TROUGH
Trough Depth = 43.555 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 1882.000 psf
FoS Against Flotation = Inf

WALL BASE SECTION
RC SECTION DESIGN
Bending Checks
Required capacity = 404.961 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.38	0.00	0.00
Smaller Bar	1.38	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.062	2.250
Area (sq-in)	2.970	0.000

Neutral Axis Depth = 4.367 in
Section is Tension controlled.
Reinforcement Strain 1.902e-02
Compression-block depth = 3.494 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 405.138 kip-ft/ft
Shear Checks
Required capacity = 72.530 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.589 sq in/ft run
Shear Capacity Provided = 170.265 kip

WALL PART-HEIGHT SECTION
RC SECTION DESIGN
Bending Checks
Required capacity = 202.481 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

ROOF MIDSPAN SECTION
RC SECTION DESIGN
Bending Checks
Required capacity = 547.547 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.62	0.00	0.00
Smaller Bar	1.62	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.938	2.250
Area (sq-in)	4.148	0.000

Neutral Axis Depth = 6.100 in
Section is Tension controlled.
Reinforcement Strain 1.271e-02
Compression-block depth = 4.880 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 550.586 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 37.912 kip

ROOF ROOT SECTION
RC SECTION DESIGN
Bending Checks
Required capacity = 1095.093 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	2.00	1.75	0.00
Smaller Bar	2.00	1.75	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	30.070	2.250
Area (sq-in)	11.094	0.000

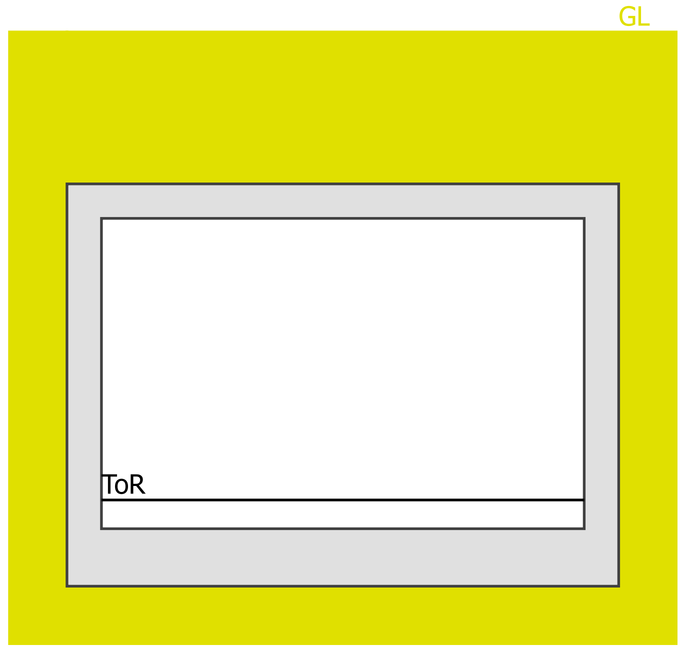
Neutral Axis Depth = 16.314 in
Section is in transition to Compression Control.
Reinforcement Strain 2.838e-03
Compression-block depth = 13.051 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 1175.358 kip-ft/ft
Shear Checks
Required capacity = 146.012 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.589 sq in/ft run
Shear Capacity Provided = 159.683 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation Preliminary Design

Section at 10921+ 0.000
Original Ground Level 288.350
Groundwater Level 235.000
Top of Rail 247.632
Top of Base 245.132
Founding Level 240.132



COVERED TROUGH
Trough Depth = 43.218 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 1882.000 psf
FoS Against Flotation = Inf

WALL BASE SECTION
RC SECTION DESIGN
Bending Checks
Required capacity = 405.685 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.50 0.00 0.00
Smaller Bar 1.50 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.000 2.250
Area (sq-in) 3.534 0.000
Neutral Axis Depth = 5.197 in
Section is Tension controlled.
Reinforcement Strain 1.547e-02
Compression-block depth = 4.158 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 475.873 kip-ft/ft
Shear Checks
Required capacity = 72.660 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.589 sq in/ft run
Shear Capacity Provided = 169.934 kip

WALL PART-HEIGHT SECTION
RC SECTION DESIGN
Bending Checks
Required capacity = 202.842 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.00 0.00 0.00
Smaller Bar 1.00 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.250 2.250
Area (sq-in) 1.571 0.000
Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

ROOF MIDSPAN SECTION
RC SECTION DESIGN
Bending Checks
Required capacity = 542.403 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.62 0.00 0.00
Smaller Bar 1.62 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 31.938 2.250
Area (sq-in) 4.148 0.000
Neutral Axis Depth = 6.100 in
Section is Tension controlled.
Reinforcement Strain 1.271e-02
Compression-block depth = 4.880 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 550.586 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 37.912 kip

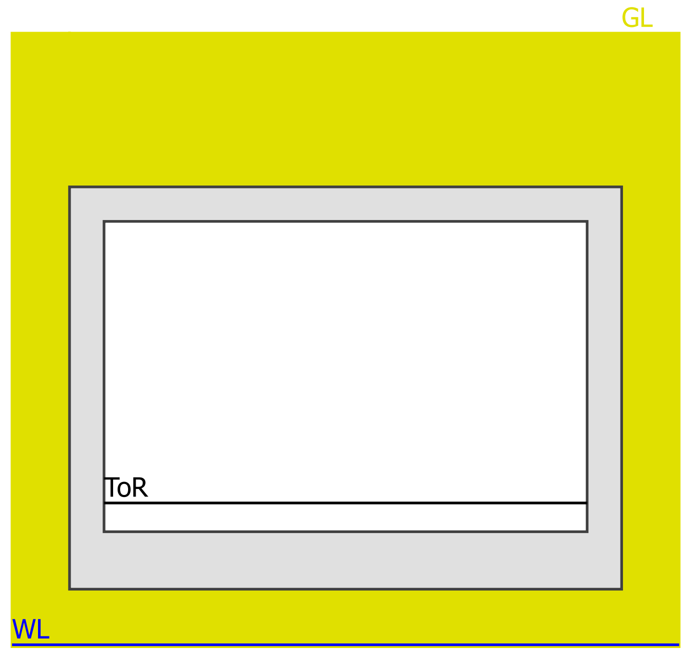
ROOF ROOT SECTION
RC SECTION DESIGN
Bending Checks
Required capacity = 1084.806 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 2.00 1.50 0.00
Smaller Bar 2.00 1.50 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 30.400 2.250
Area (sq-in) 9.817 0.000
Neutral Axis Depth = 14.437 in
Section is in transition to Compression Control.
Reinforcement Strain 3.597e-03
Compression-block depth = 11.550 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 1087.900 kip-ft/ft
Shear Checks
Required capacity = 144.641 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.589 sq in/ft run
Shear Capacity Provided = 161.437 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10921+50.000
Original Ground Level 288.210
Groundwater Level 235.000
Top of Rail 247.335
Top of Base 244.835
Founding Level 239.835



COVERED TROUGH
Trough Depth = 43.375 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 1882.000 psf
FoS Against Flotation = Inf

WALL BASE SECTION
RC SECTION DESIGN
Bending Checks
Required capacity = 410.181 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.50	0.00	0.00
Smaller Bar	1.50	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.000	2.250
Area (sq-in)	3.534	0.000

Neutral Axis Depth = 5.197 in
Section is Tension controlled.
Reinforcement Strain 1.547e-02
Compression-block depth = 4.158 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 475.873 kip-ft/ft
Shear Checks
Required capacity = 73.465 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.589 sq in/ft run
Shear Capacity Provided = 169.934 kip

WALL PART-HEIGHT SECTION
RC SECTION DESIGN
Bending Checks
Required capacity = 205.091 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

ROOF MIDSPAN SECTION
RC SECTION DESIGN
Bending Checks
Required capacity = 544.797 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.62	0.00	0.00
Smaller Bar	1.62	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.938	2.250
Area (sq-in)	4.148	0.000

Neutral Axis Depth = 6.100 in
Section is Tension controlled.
Reinforcement Strain 1.271e-02
Compression-block depth = 4.880 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 550.586 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 37.912 kip

ROOF ROOT SECTION
RC SECTION DESIGN
Bending Checks
Required capacity = 1089.594 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	2.00	1.75	0.00
Smaller Bar	2.00	1.75	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	30.070	2.250
Area (sq-in)	11.094	0.000

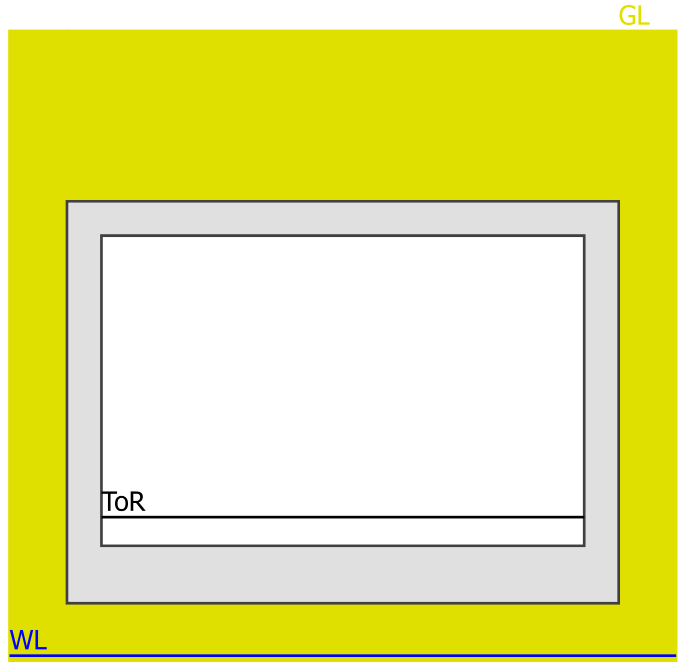
Neutral Axis Depth = 16.314 in
Section is in transition to Compression Control.
Reinforcement Strain 2.838e-03
Compression-block depth = 13.051 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 1175.358 kip-ft/ft
Shear Checks
Required capacity = 145.279 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.589 sq in/ft run
Shear Capacity Provided = 159.683 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10922+ 0.000
Original Ground Level 289.390
Groundwater Level 235.000
Top of Rail 247.065
Top of Base 244.565
Founding Level 239.565



COVERED TROUGH
Trough Depth = 44.825 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 1882.000 psf
FoS Against Flotation = Inf

WALL BASE SECTION
RC SECTION DESIGN
Bending Checks
Required capacity = 425.021 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.50	0.00	0.00
Smaller Bar	1.50	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.000	2.250
Area (sq-in)	3.534	0.000

Neutral Axis Depth = 5.197 in
Section is Tension controlled.
Reinforcement Strain 1.547e-02
Compression-block depth = 4.158 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 475.873 kip-ft/ft
Shear Checks
Required capacity = 76.123 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.589 sq in/ft run
Shear Capacity Provided = 169.934 kip

WALL PART-HEIGHT SECTION
RC SECTION DESIGN
Bending Checks
Required capacity = 212.511 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

ROOF MIDSPAN SECTION
RC SECTION DESIGN
Bending Checks
Required capacity = 566.937 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.75	0.00	0.00
Smaller Bar	1.75	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.875	2.250
Area (sq-in)	4.811	0.000

Neutral Axis Depth = 7.074 in
Section is Tension controlled.
Reinforcement Strain 1.052e-02
Compression-block depth = 5.659 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 628.758 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 37.838 kip

ROOF ROOT SECTION
RC SECTION DESIGN
Bending Checks
Required capacity = 1133.874 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	2.00	1.75	0.00
Smaller Bar	2.00	1.75	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	30.070	2.250
Area (sq-in)	11.094	0.000

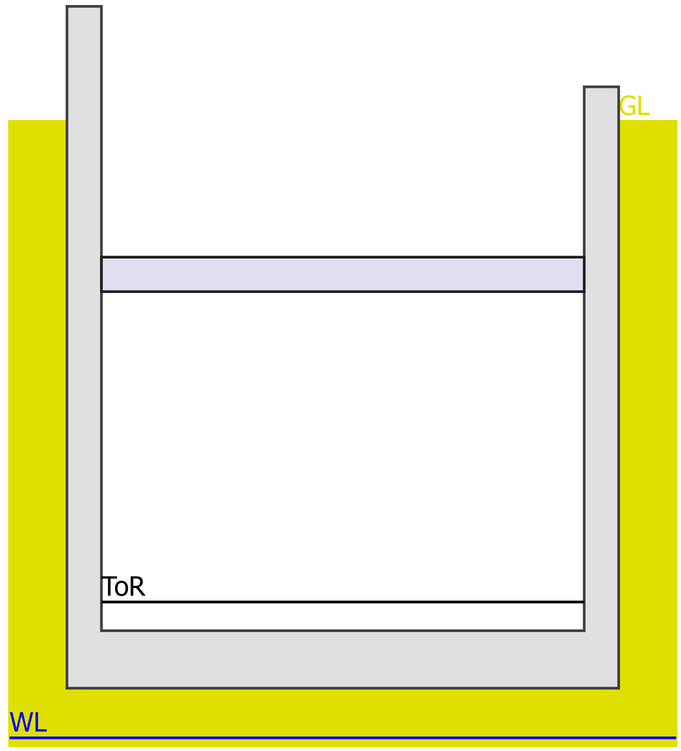
Neutral Axis Depth = 16.314 in
Section is in transition to Compression Control.
Reinforcement Strain 2.838e-03
Compression-block depth = 13.051 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 1175.358 kip-ft/ft
Shear Checks
Required capacity = 151.183 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.589 sq in/ft run
Shear Capacity Provided = 159.683 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10922+50.000
Original Ground Level 288.640
Groundwater Level 235.000
Top of Rail 246.820
Top of Base 244.320
Founding Level 239.320



BRACED U-TROUGH
Trough Depth = 44.320 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf

BASE OF WALL AT 44.320 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 120.464 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.00 0.00 0.00
Smaller Bar 1.00 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.250 2.250
Area (sq-in) 1.571 0.000
Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 10.668 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL AT 15.083 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 74.328 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.00 0.00 0.00
Smaller Bar 1.00 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.250 2.250
Area (sq-in) 1.571 0.000
Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL PROP
Prop Spacing = 10.000 ft
Prop Force = 34.113 kip/ft
Force per prop = 341.132 kips

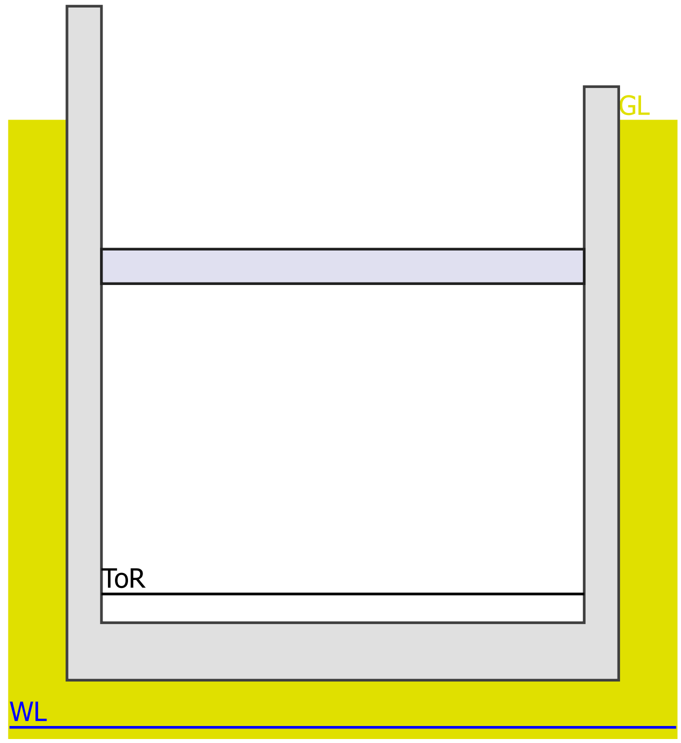
WALL AT PROP LEVEL (13.320 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 294.174 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.25 0.00 0.00
Smaller Bar 1.25 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.125 2.250
Area (sq-in) 2.454 0.000
Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 23.895 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10923+ 0.000
Original Ground Level 287.740
Groundwater Level 235.000
Top of Rail 246.602
Top of Base 244.102
Founding Level 239.102



BRACED U-TROUGH
Trough Depth = 43.638 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf

BASE OF WALL AT 43.638 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 117.360 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 10.610 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL AT 15.511 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 80.530 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL PROP
Prop Spacing = 10.000 ft
Prop Force = 34.329 kip/ft
Force per prop = 343.293 kips

WALL AT PROP LEVEL (12.638 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 290.790 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

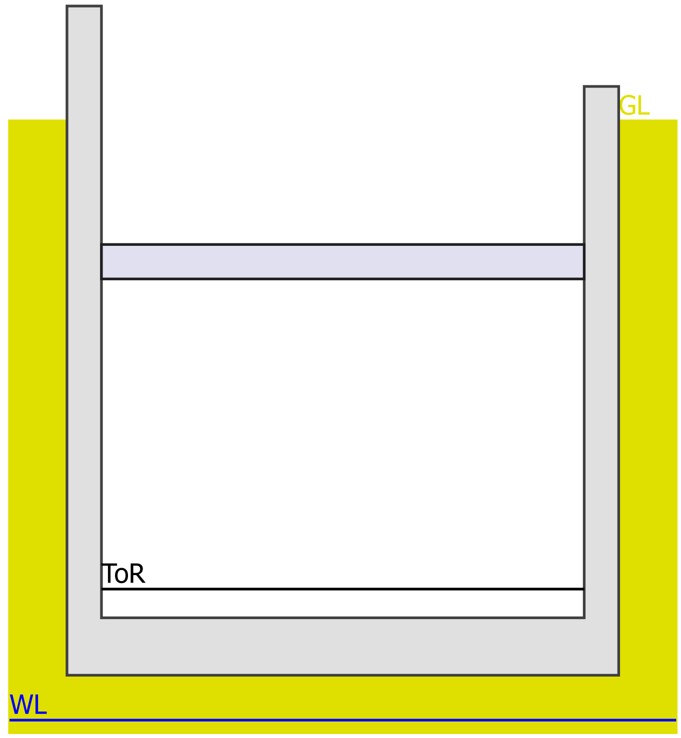
Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.125	2.250
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 23.460 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

Section at 10923+50.000
Original Ground Level 287.170
Groundwater Level 235.000
Top of Rail 246.409
Top of Base 243.909
Founding Level 238.909



BRACED U-TROUGH
Trough Depth = 43.261 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf

BASE OF WALL AT 43.261 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 115.278 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 10.506 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL AT 15.738 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 83.856 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL PROP
Prop Spacing = 10.000 ft
Prop Force = 34.474 kip/ft
Force per prop = 344.743 kips

WALL AT PROP LEVEL (12.261 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 288.992 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.125	2.250
Area (sq-in)	2.454	0.000

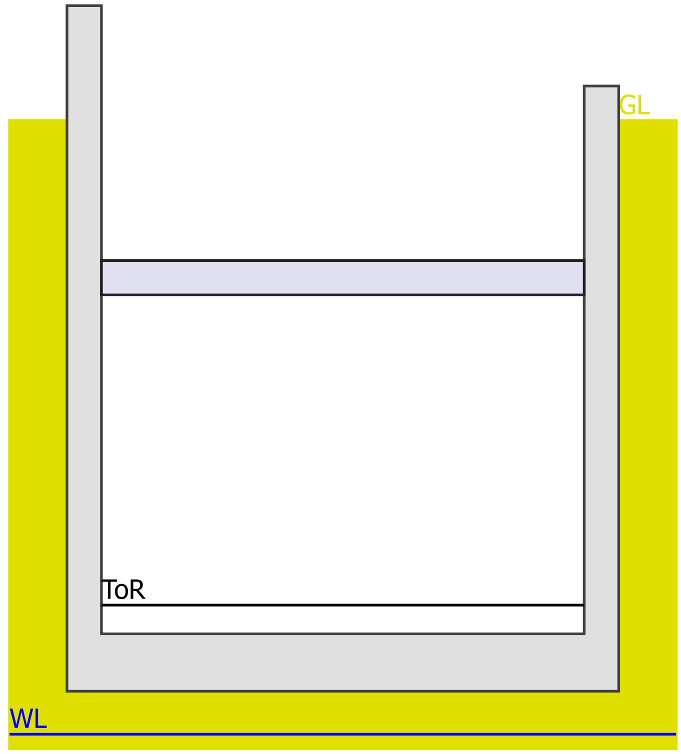
Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 23.219 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10924+ 0.000
Original Ground Level 288.430
Groundwater Level 235.000
Top of Rail 246.243
Top of Base 243.743
Founding Level 238.743



BRACED U-TROUGH
Trough Depth = 44.687 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf

BASE OF WALL AT 44.687 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 117.841 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.00 0.00 0.00
Smaller Bar 1.00 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.250 2.250
Area (sq-in) 1.571 0.000
Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 10.002 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL AT 14.843 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 70.884 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.00 0.00 0.00
Smaller Bar 1.00 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.250 2.250
Area (sq-in) 1.571 0.000
Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL PROP
Prop Spacing = 10.000 ft
Prop Force = 34.019 kip/ft
Force per prop = 340.194 kips

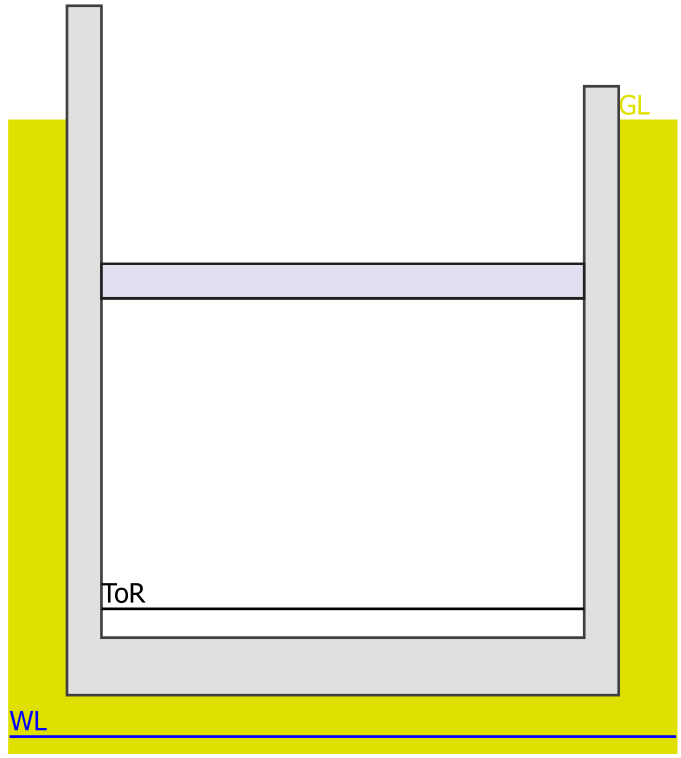
WALL AT PROP LEVEL (13.687 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 296.069 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.25 0.00 0.00
Smaller Bar 1.25 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.125 2.250
Area (sq-in) 2.454 0.000
Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 24.100 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10924+50.000
Original Ground Level 288.540
Groundwater Level 235.000
Top of Rail 246.103
Top of Base 243.603
Founding Level 238.603



BRACED U-TROUGH
Trough Depth = 44.937 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf

BASE OF WALL AT 44.937 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 117.710 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.00 0.00 0.00
Smaller Bar 1.00 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.250 2.250
Area (sq-in) 1.571 0.000
Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 9.803 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL AT 14.676 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 68.497 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.00 0.00 0.00
Smaller Bar 1.00 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.250 2.250
Area (sq-in) 1.571 0.000
Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL PROP
Prop Spacing = 10.000 ft
Prop Force = 33.964 kip/ft
Force per prop = 339.639 kips

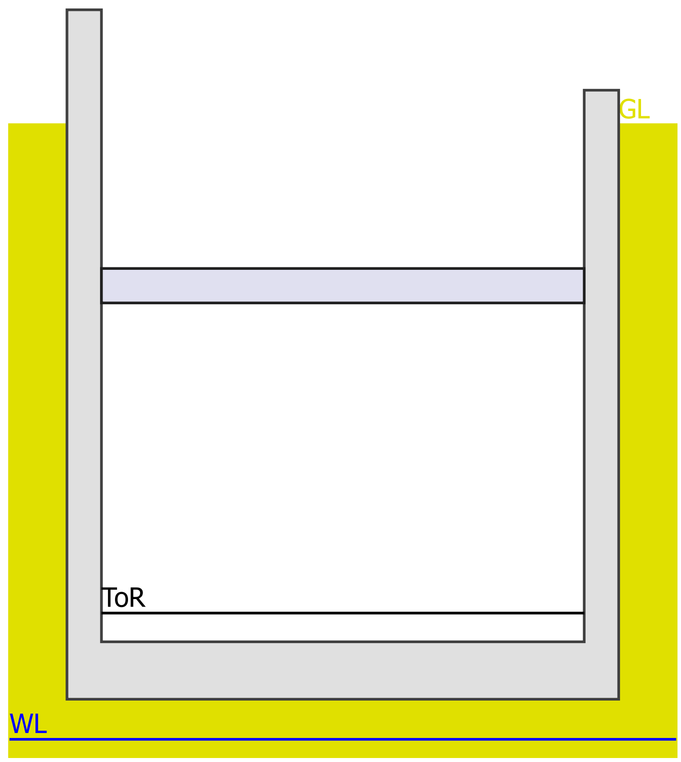
WALL AT PROP LEVEL (13.937 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 297.389 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.25 0.00 0.00
Smaller Bar 1.25 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.125 2.250
Area (sq-in) 2.454 0.000
Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 24.254 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10925+ 0.000
Original Ground Level 288.510
Groundwater Level 235.000
Top of Rail 245.989
Top of Base 243.489
Founding Level 238.489



BRACED U-TROUGH
Trough Depth = 45.021 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf

BASE OF WALL AT 45.021 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 117.312 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 9.674 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL AT 14.619 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 67.689 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL PROP
Prop Spacing = 10.000 ft
Prop Force = 33.947 kip/ft
Force per prop = 339.467 kips

WALL AT PROP LEVEL (14.021 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 297.838 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.125	2.250
Area (sq-in)	2.454	0.000

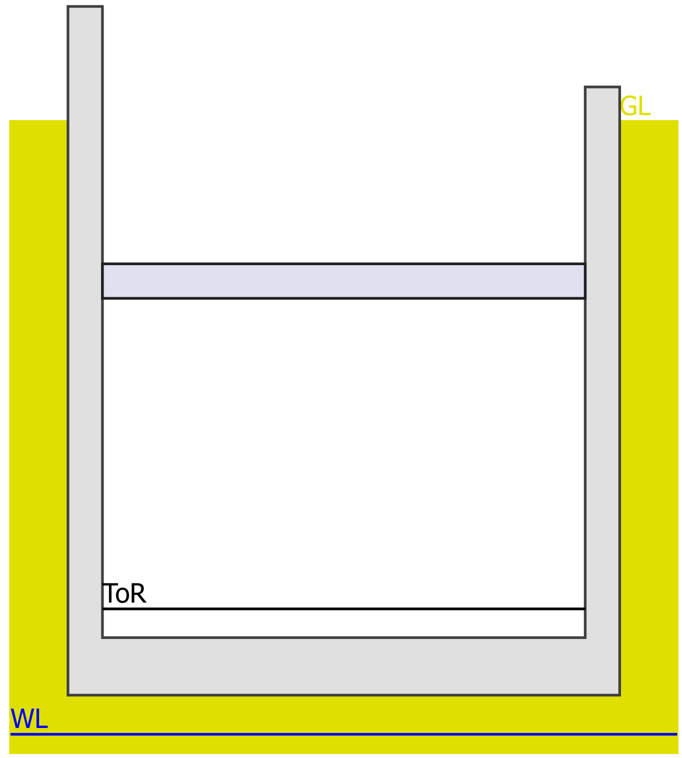
Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 24.303 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10925+50.000
Original Ground Level 288.280
Groundwater Level 235.000
Top of Rail 245.901
Top of Base 243.401
Founding Level 238.401



BRACED U-TROUGH
Trough Depth = 44.879 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf

BASE OF WALL AT 44.879 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 116.466 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 9.628 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL AT 14.715 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 69.057 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL AT PROP LEVEL (13.879 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 297.079 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

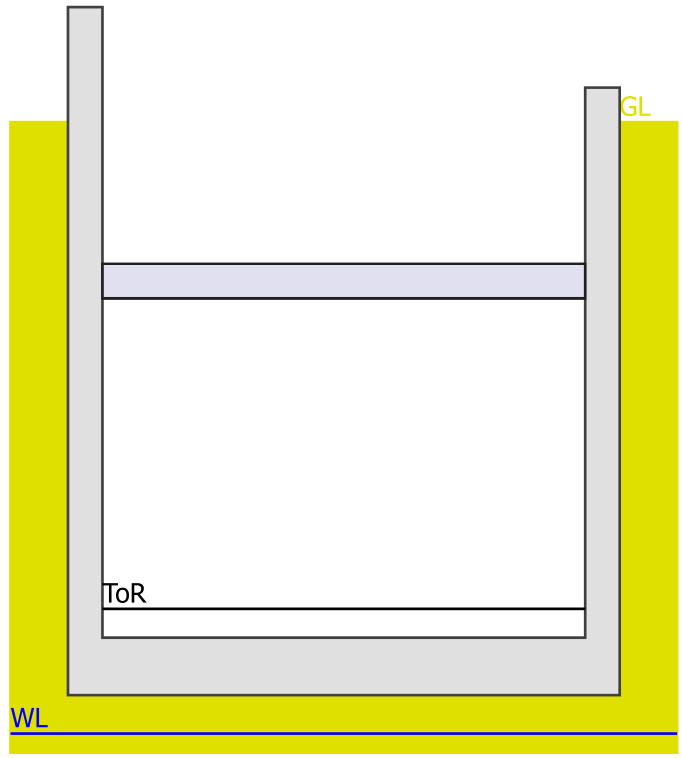
Layer	Tension	Compression
Depth (in)	32.125	2.250
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 24.208 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

WALL PROP
Prop Spacing = 10.000 ft
Prop Force = 33.976 kip/ft
Force per prop = 339.762 kips

Fresno Grade Separation Preliminary Design

Section at 10926+ 0.000
Original Ground Level 288.160
Groundwater Level 235.000
Top of Rail 245.840
Top of Base 243.340
Founding Level 238.340



BRACED U-TROUGH
Trough Depth = 44.820 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf

BASE OF WALL AT 44.820 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 115.990 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.00 0.00 0.00
Smaller Bar 1.00 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.250 2.250
Area (sq-in) 1.571 0.000
Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 9.586 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL AT 14.754 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 69.616 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.00 0.00 0.00
Smaller Bar 1.00 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.250 2.250
Area (sq-in) 1.571 0.000
Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL PROP
Prop Spacing = 10.000 ft
Prop Force = 33.989 kip/ft
Force per prop = 339.890 kips

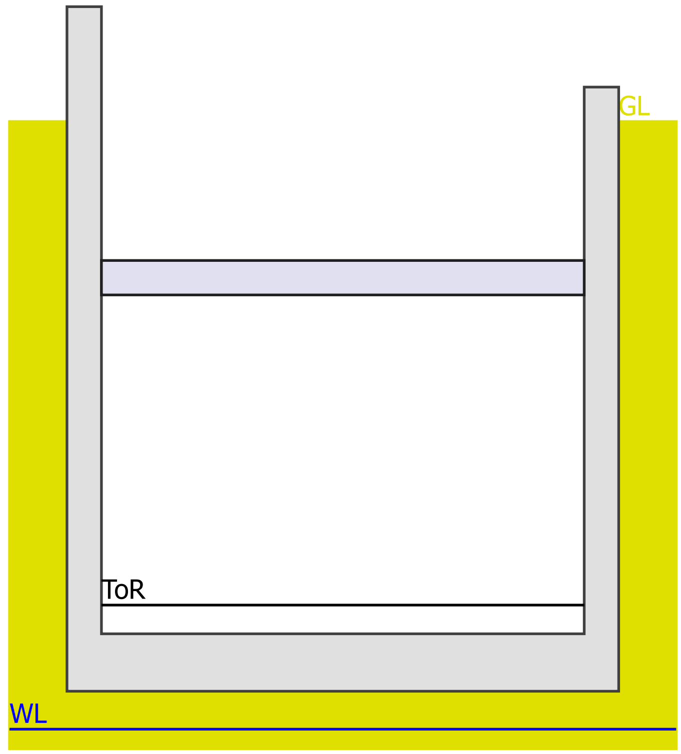
WALL AT PROP LEVEL (13.820 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 296.770 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.25 0.00 0.00
Smaller Bar 1.25 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.125 2.250
Area (sq-in) 2.454 0.000
Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 24.169 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10926+50.000
Original Ground Level 287.900
Groundwater Level 235.000
Top of Rail 245.804
Top of Base 243.304
Founding Level 238.304



BRACED U-TROUGH
Trough Depth = 44.596 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf

BASE OF WALL AT 44.596 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 115.218 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.00 0.00 0.00
Smaller Bar 1.00 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.250 2.250
Area (sq-in) 1.571 0.000
Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 9.610 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL AT 14.903 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 71.747 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.00 0.00 0.00
Smaller Bar 1.00 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.250 2.250
Area (sq-in) 1.571 0.000
Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL PROP
Prop Spacing = 10.000 ft
Prop Force = 34.041 kip/ft
Force per prop = 340.413 kips

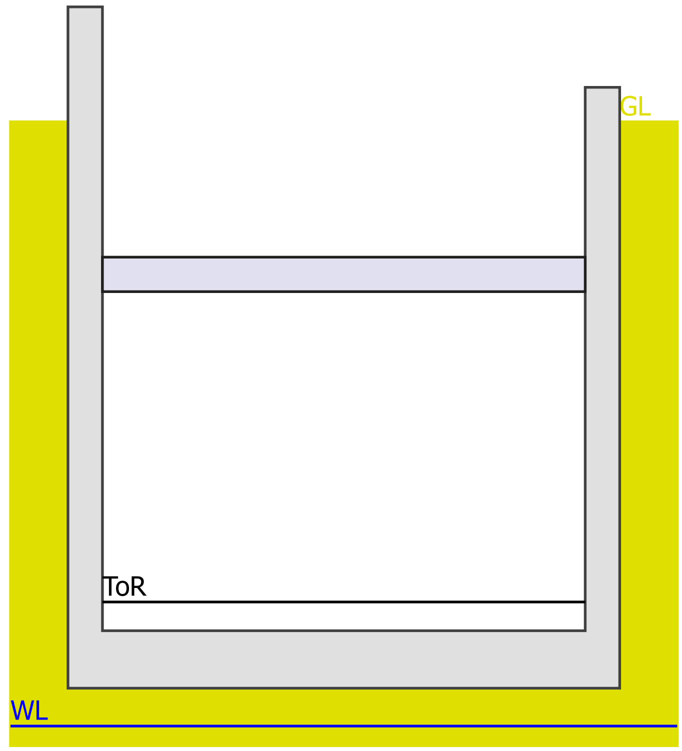
WALL AT PROP LEVEL (13.596 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 295.592 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.25 0.00 0.00
Smaller Bar 1.25 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.125 2.250
Area (sq-in) 2.454 0.000
Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 24.024 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10927+ 0.000
Original Ground Level 287.570
Groundwater Level 235.000
Top of Rail 245.795
Top of Base 243.295
Founding Level 238.295



BRACED U-TROUGH
Trough Depth = 44.275 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf

BASE OF WALL AT 44.275 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 114.347 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 9.681 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL AT 15.112 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 74.745 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL PROP
Prop Spacing = 10.000 ft
Prop Force = 34.126 kip/ft
Force per prop = 341.258 kips

WALL AT PROP LEVEL (13.275 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 293.945 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

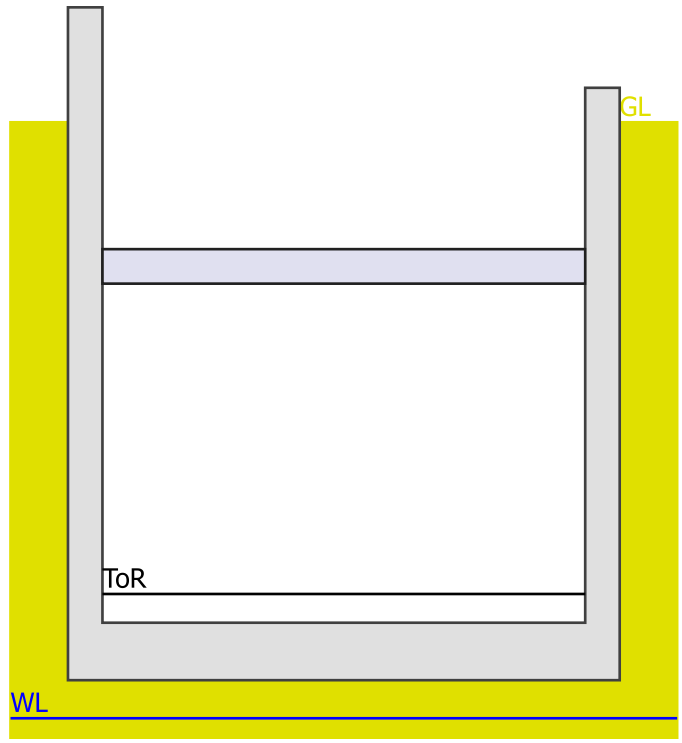
Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.125	2.250
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 23.822 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

Fresno Grade Separation
Preliminary Design

Section at 10927+50.000
Original Ground Level 286.850
Groundwater Level 235.000
Top of Rail 245.812
Top of Base 243.312
Founding Level 238.312



BRACED U-TROUGH
Trough Depth =   43.538 ft
Base Thickness =   5.000 ft
Min Wall Thickness =   3.000 ft
Trough internal width =   42.000 ft
Design Load factors
DL   EH   WA   LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 420.000 psf
FoS Against Flotation =   Inf

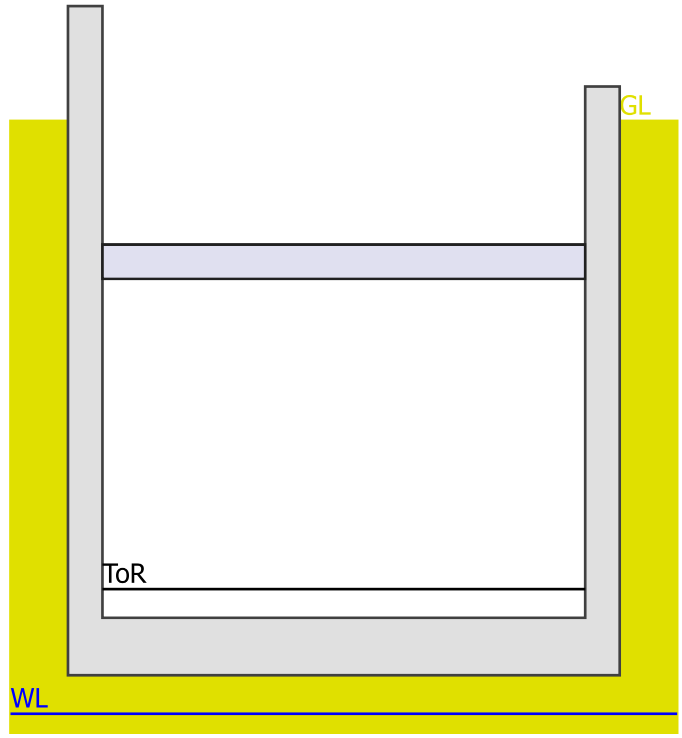
BASE OF WALL AT 43.538 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 112.614 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer   T1   T2   C1
Larger Bar   1.00 0.00 0.00
Smaller Bar   1.00 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer   Tension   Compression
Depth (in)   32.250   2.250
Area (sq-in)   1.571   0.000
Neutral Axis Depth =   2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth =   1.848 in
Resistance factor (Phi) =   0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity =   9.873 kip
Shear Link Spacing L =   6.000 in
Shear Link Spacing T =   6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area =   0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL AT 15.572 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 81.421 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer   T1   T2   C1
Larger Bar   1.00 0.00 0.00
Smaller Bar   1.00 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer   Tension   Compression
Depth (in)   32.250   2.250
Area (sq-in)   1.571   0.000
Neutral Axis Depth =   2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth =   1.848 in
Resistance factor (Phi) =   0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity =   0.000 kip
Shear Link Spacing L =   6.000 in
Shear Link Spacing T =   6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area =   0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL AT PROP LEVEL (12.538 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 290.308 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer   T1   T2   C1
Larger Bar   1.25 0.00 0.00
Smaller Bar   1.25 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer   Tension   Compression
Depth (in)   32.125   2.250
Area (sq-in)   2.454   0.000
Neutral Axis Depth =   3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth =   2.887 in
Resistance factor (Phi) =   0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity =   23.365 kip
Shear Link Spacing L =   6.000 in
Shear Link Spacing T =   6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area =   0.000 sq in/ft run
Shear Capacity Provided =   38.135 kip

WALL PROP
Prop Spacing =   10.000 ft
Prop Force =   34.366 kip/ft
Force per prop = 343.660 kips

Section at 10928+ 0.000
Original Ground Level 286.610
Groundwater Level 235.000
Top of Rail 245.855
Top of Base 243.355
Founding Level 238.355



BRACED U-TROUGH
Trough Depth = 43.255 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf

BASE OF WALL AT 43.255 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 112.165 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 9.977 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL AT 15.741 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 83.903 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL PROP
Prop Spacing = 10.000 ft
Prop Force = 34.477 kip/ft
Force per prop = 344.766 kips

WALL AT PROP LEVEL (12.255 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 288.966 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

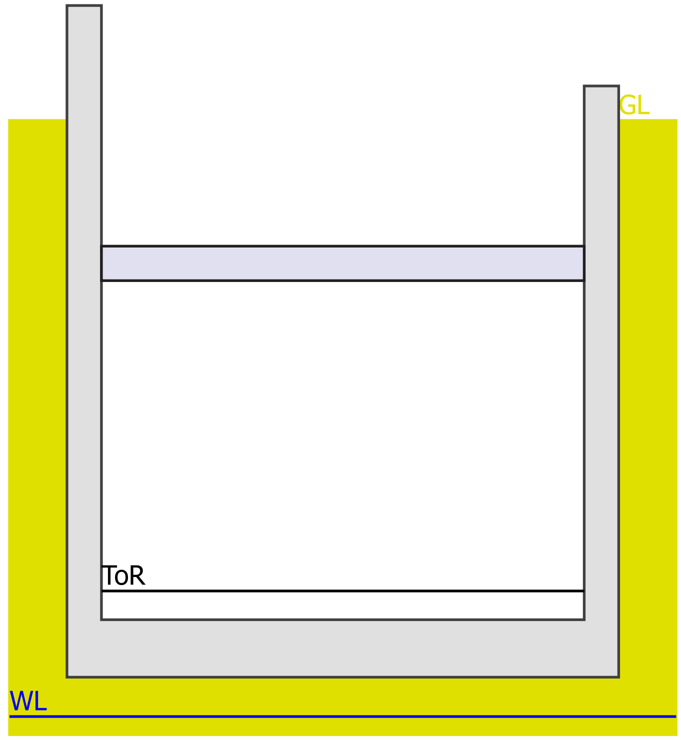
Layer	Tension	Compression
Depth (in)	32.125	2.250
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 23.193 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation Preliminary Design

Section at 10928+50.000
Original Ground Level 286.860
Groundwater Level 235.000
Top of Rail 245.924
Top of Base 243.424
Founding Level 238.424



BRACED U-TROUGH
Trough Depth = 43.436 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf

BASE OF WALL AT 43.436 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 112.954 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.00 0.00 0.00
Smaller Bar 1.00 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.250 2.250
Area (sq-in) 1.571 0.000
Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 10.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL AT 15.633 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 82.321 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.00 0.00 0.00
Smaller Bar 1.00 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.250 2.250
Area (sq-in) 1.571 0.000
Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL PROP
Prop Spacing = 10.000 ft
Prop Force = 34.405 kip/ft
Force per prop = 344.046 kips

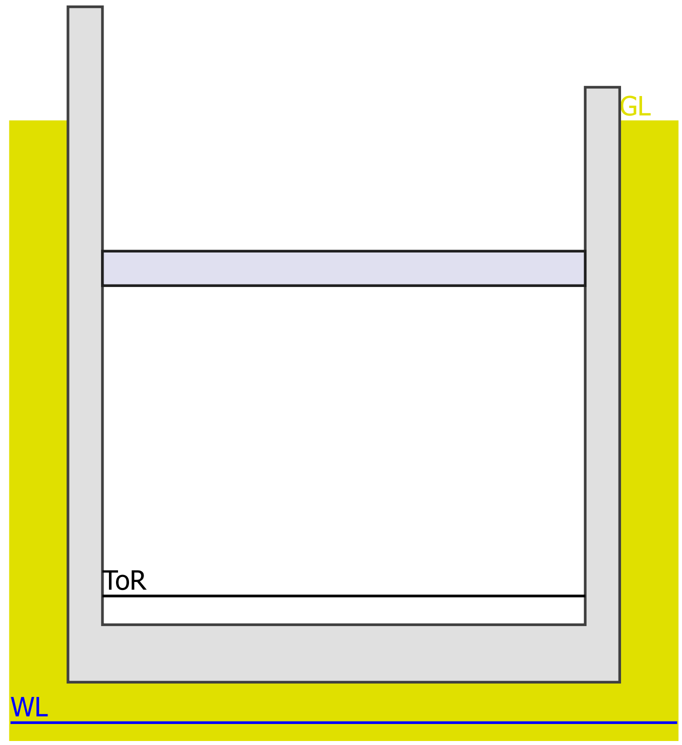
WALL AT PROP LEVEL (12.436 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 289.820 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.25 0.00 0.00
Smaller Bar 1.25 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.125 2.250
Area (sq-in) 2.454 0.000
Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 23.306 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10929+ 0.000
Original Ground Level 287.280
Groundwater Level 235.000
Top of Rail 246.019
Top of Base 243.519
Founding Level 238.519



BRACED U-TROUGH
Trough Depth = 43.761 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf

BASE OF WALL AT 43.761 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 114.247 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 10.014 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL AT 15.435 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 79.434 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL PROP
Prop Spacing = 10.000 ft
Prop Force = 34.286 kip/ft
Force per prop = 342.862 kips

WALL AT PROP LEVEL (12.761 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 291.385 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.125	2.250
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 23.510 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

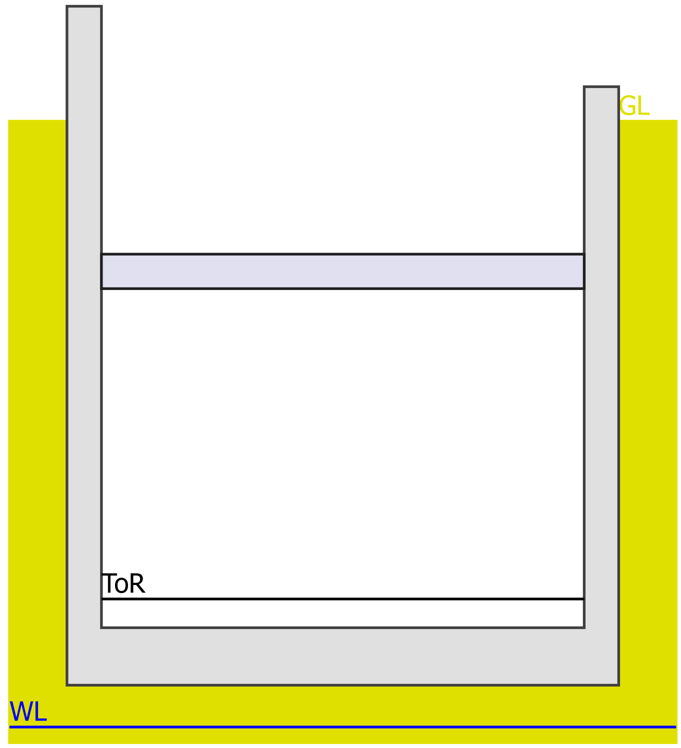
URS | HMM | ARUP
CALIFORNIA HIGH-SPEED TRAIN

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10929+50.000
Original Ground Level 287.710
Groundwater Level 235.000
Top of Rail 246.140
Top of Base 243.640
Founding Level 238.640



BRACED U-TROUGH
Trough Depth = 44.070 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf

BASE OF WALL AT 44.070 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 115.678 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.00 0.00 0.00
Smaller Bar 1.00 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.250 2.250
Area (sq-in) 1.571 0.000
Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 10.056 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL AT 15.242 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 76.636 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.00 0.00 0.00
Smaller Bar 1.00 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.250 2.250
Area (sq-in) 1.571 0.000
Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

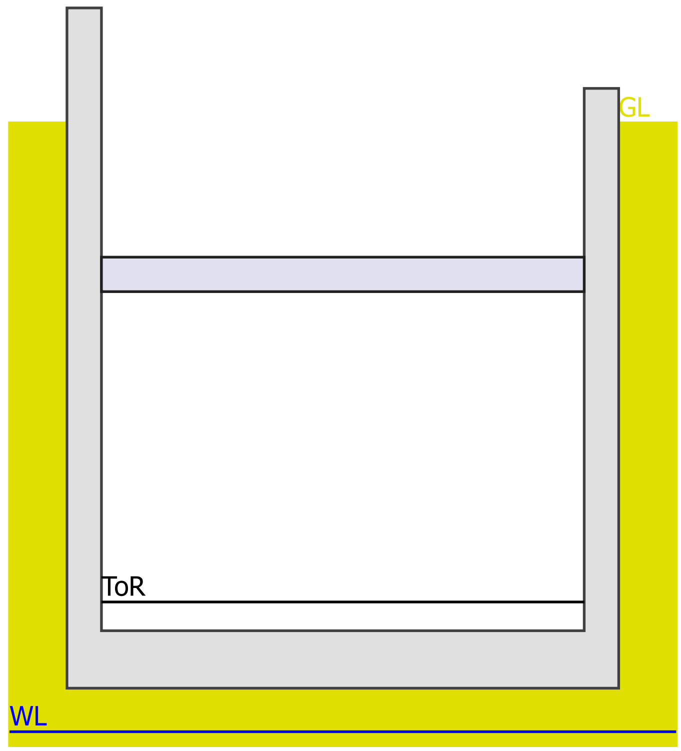
WALL PROP
Prop Spacing = 10.000 ft
Prop Force = 34.186 kip/ft
Force per prop = 341.861 kips

WALL AT PROP LEVEL (13.070 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 292.910 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.25 0.00 0.00
Smaller Bar 1.25 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.125 2.250
Area (sq-in) 2.454 0.000
Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 23.707 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation Preliminary Design

Section at 10930+ 0.000
Original Ground Level 287.970
Groundwater Level 235.000
Top of Rail 246.288
Top of Base 243.788
Founding Level 238.788



BRACED U-TROUGH
Trough Depth = 44.182 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 420.000 psf
FoS Against Flotation = Inf

BASE OF WALL AT 44.182 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 116.803 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 10.171 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL AT 15.171 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 75.602 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL PROP
Prop Spacing = 10.000 ft
Prop Force = 34.152 kip/ft
Force per prop = 341.524 kips

WALL AT PROP LEVEL (13.182 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 293.475 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.125	2.250
Area (sq-in)	2.454	0.000

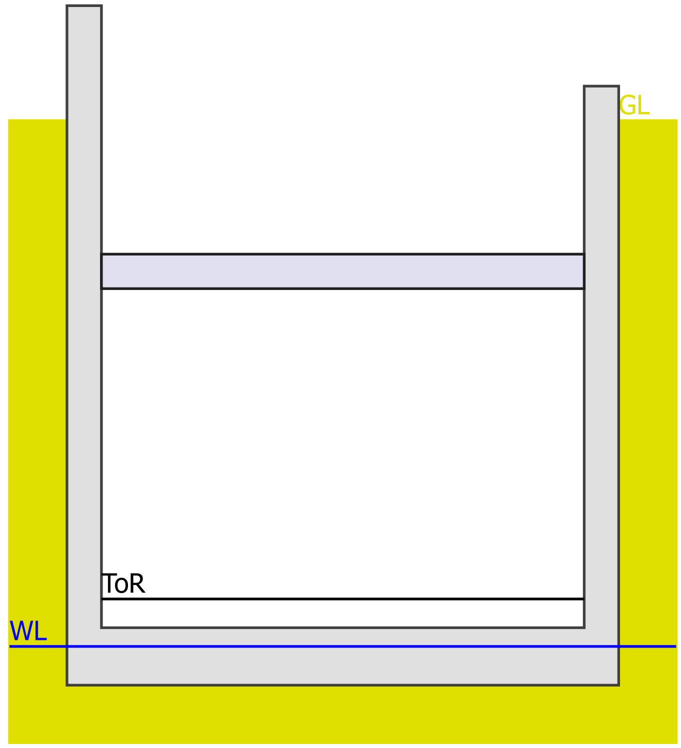
Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 23.783 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10930+50.000
Original Ground Level 288.080
Groundwater Level 242.333
Top of Rail 246.461
Top of Base 243.961
Founding Level 238.961



BRACED U-TROUGH
Trough Depth = 44.119 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 420.000 psf
FoS Against Flotation = 8.398

BASE OF WALL AT 44.119 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 100.719 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 6.621 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL AT 15.211 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 76.187 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL PROP
Prop Spacing = 10.000 ft
Prop Force = 34.171 kip/ft
Force per prop = 341.712 kips

WALL AT PROP LEVEL (13.119 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 293.155 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

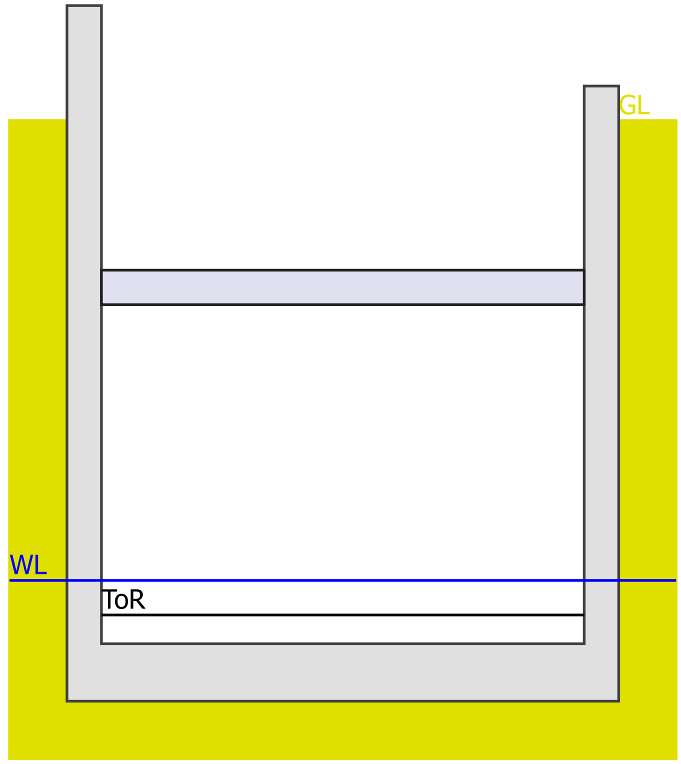
Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.125	2.250
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 23.649 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

Fresno Grade Separation
Preliminary Design

Section at 10931+ 0.000
Original Ground Level 289.680
Groundwater Level 249.667
Top of Rail 246.661
Top of Base 244.161
Founding Level 239.161



BRACED U-TROUGH
Trough Depth = 45.519 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = 2.737

BASE OF WALL AT 45.519 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 101.532 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 7.198 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL AT 14.275 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 62.818 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL PROP
Prop Spacing = 10.000 ft
Prop Force = 33.860 kip/ft
Force per prop = 338.596 kips

WALL AT PROP LEVEL (14.519 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 300.556 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.125	2.250
Area (sq-in)	2.454	0.000

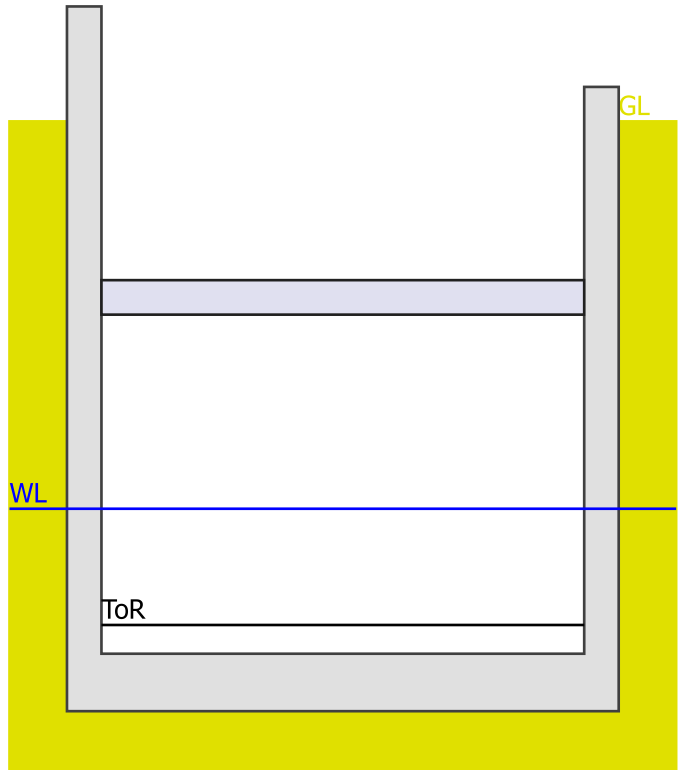
Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 24.556 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10931+50.000
Original Ground Level 290.700
Groundwater Level 257.000
Top of Rail 246.887
Top of Base 244.387
Founding Level 239.387



BRACED U-TROUGH
Trough Depth = 46.313 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 420.000 psf
FoS Against Flotation = 1.647

BASE OF WALL AT 46.313 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 78.782 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.00 0.00 0.00
Smaller Bar 1.00 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.250 2.250
Area (sq-in) 1.571 0.000
Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 14.084 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL AT 13.702 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 54.800 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.00 0.00 0.00
Smaller Bar 1.00 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.250 2.250
Area (sq-in) 1.571 0.000
Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

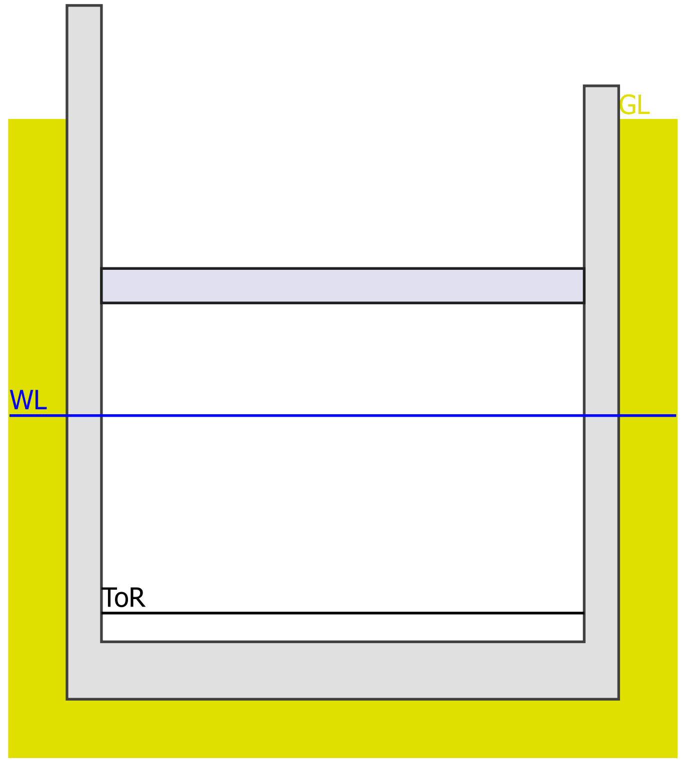
WALL AT PROP LEVEL (15.313 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 305.086 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.25 0.00 0.00
Smaller Bar 1.25 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.125 2.250
Area (sq-in) 2.454 0.000
Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 25.415 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

WALL PROP
Prop Spacing = 10.000 ft
Prop Force = 33.770 kip/ft
Force per prop = 337.695 kips

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10932+ 0.000
Original Ground Level 290.040
Groundwater Level 264.333
Top of Rail 247.139
Top of Base 244.639
Founding Level 239.639



BRACED U-TROUGH
Trough Depth = 45.401 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 420.000 psf
FoS Against Flotation = 1.163
Additional tie down force required 24.927 kips

BASE OF WALL AT 45.401 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 112.056 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.00 0.00 0.00
Smaller Bar 1.00 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.250 2.250
Area (sq-in) 1.571 0.000
Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 25.931 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL AT 14.358 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 63.983 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.00 0.00 0.00
Smaller Bar 1.00 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.250 2.250
Area (sq-in) 1.571 0.000
Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL PROP
Prop Spacing = 10.000 ft
Prop Force = 33.878 kip/ft
Force per prop = 338.780 kips

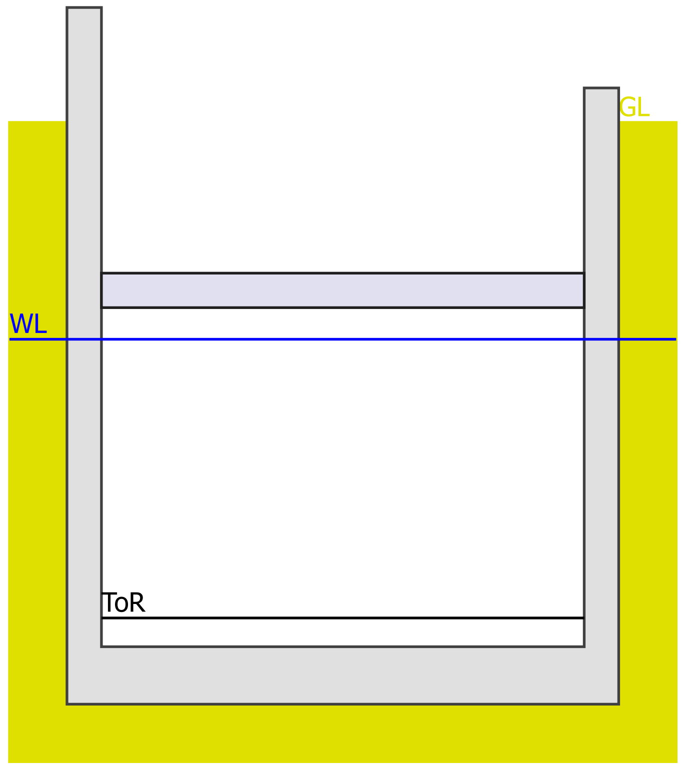
WALL AT PROP LEVEL (14.401 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 299.904 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.25 0.00 0.00
Smaller Bar 1.25 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.125 2.250
Area (sq-in) 2.454 0.000
Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 26.440 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10932+50.000
Original Ground Level 290.530
Groundwater Level 271.667
Top of Rail 247.417
Top of Base 244.917
Founding Level 239.917



BRACED U-TROUGH
Trough Depth = 45.613 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 420.000 psf
FoS Against Flotation = 0.907
Additional tie down force required 56.426 kips

BASE OF WALL AT 45.613 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 196.792 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.00 0.00 0.00
Smaller Bar 1.00 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.250 2.250
Area (sq-in) 1.571 0.000
Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 40.415 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

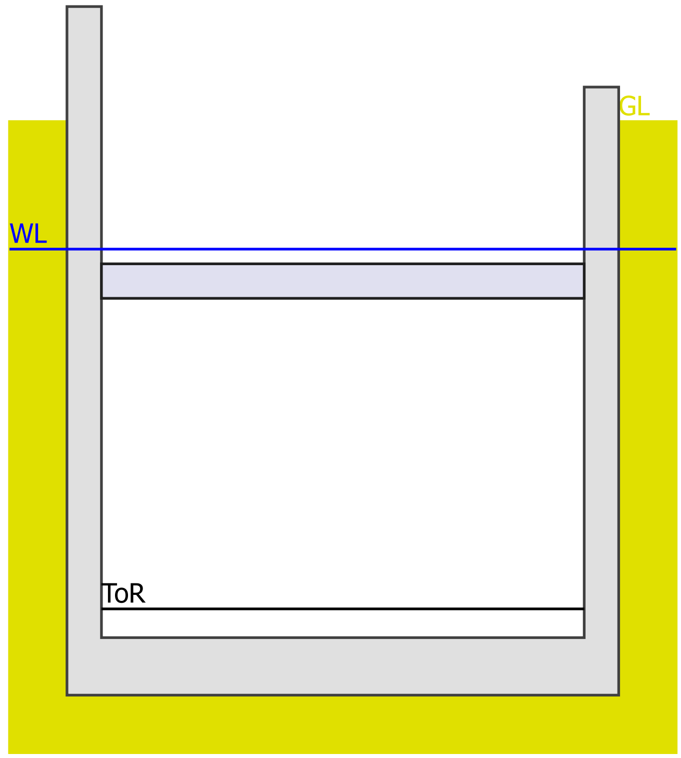
WALL AT 14.209 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 61.886 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.00 0.00 0.00
Smaller Bar 1.00 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.250 2.250
Area (sq-in) 1.571 0.000
Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL PROP
Prop Spacing = 10.000 ft
Prop Force = 33.846 kip/ft
Force per prop = 338.459 kips

WALL AT PROP LEVEL (14.613 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 301.079 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.25 0.00 0.00
Smaller Bar 1.25 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.125 2.250
Area (sq-in) 2.454 0.000
Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 30.972 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

Fresno Grade Separation
Preliminary Design

Section at 10933+ 0.000
Original Ground Level 290.090
Groundwater Level 279.000
Top of Rail 247.722
Top of Base 245.222
Founding Level 240.222



BRACED U-TROUGH
Trough Depth = 44.868 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 420.000 psf
FoS Against Flotation = 0.736
Additional tie down force required 88.703 kips

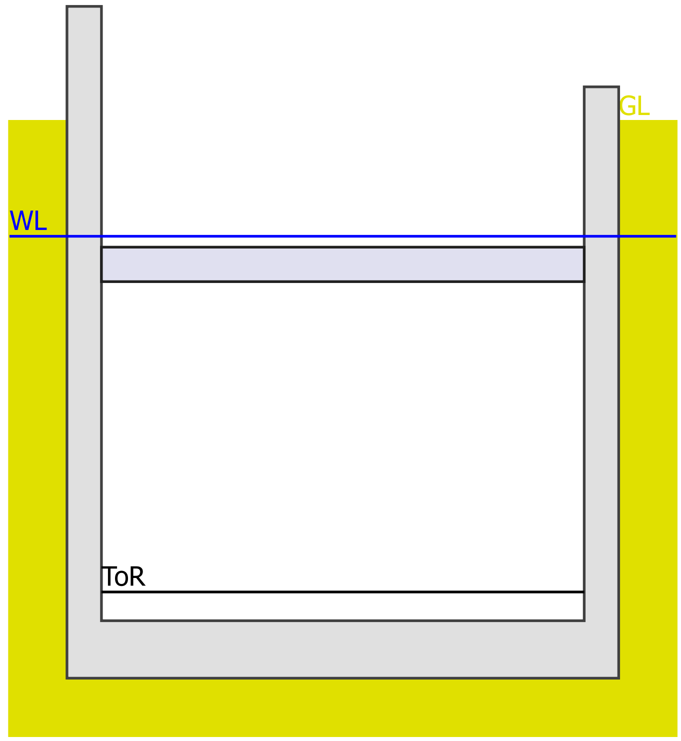
BASE OF WALL AT 44.868 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 257.098 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.12 0.00 0.00
Smaller Bar 1.12 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.188 2.250
Area (sq-in) 1.988 0.000
Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 50.059 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.589 sq in/ft run
Shear Capacity Provided = 170.929 kip

WALL AT 14.722 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 69.154 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.00 0.00 0.00
Smaller Bar 1.00 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.250 2.250
Area (sq-in) 1.571 0.000
Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL PROP
Prop Spacing = 10.000 ft
Prop Force = 33.978 kip/ft
Force per prop = 339.784 kips

WALL AT PROP LEVEL (13.868 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 297.025 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.25 0.00 0.00
Smaller Bar 1.25 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.125 2.250
Area (sq-in) 2.454 0.000
Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 35.230 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

Section at 10933+50.000
Original Ground Level 289.000
Groundwater Level 279.000
Top of Rail 248.052
Top of Base 245.552
Founding Level 240.552



BRACED U-TROUGH
Trough Depth = 43.448 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = 0.731
Additional tie down force required 88.548 kips

BASE OF WALL AT 43.448 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 260.673 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.188	2.250
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 50.405 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.589 sq in/ft run
Shear Capacity Provided = 170.929 kip

WALL AT 15.626 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 82.217 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL PROP
Prop Spacing = 10.000 ft
Prop Force = 34.400 kip/ft
Force per prop = 344.000 kips

WALL AT PROP LEVEL (12.448 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 289.877 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.125	2.250
Area (sq-in)	2.454	0.000

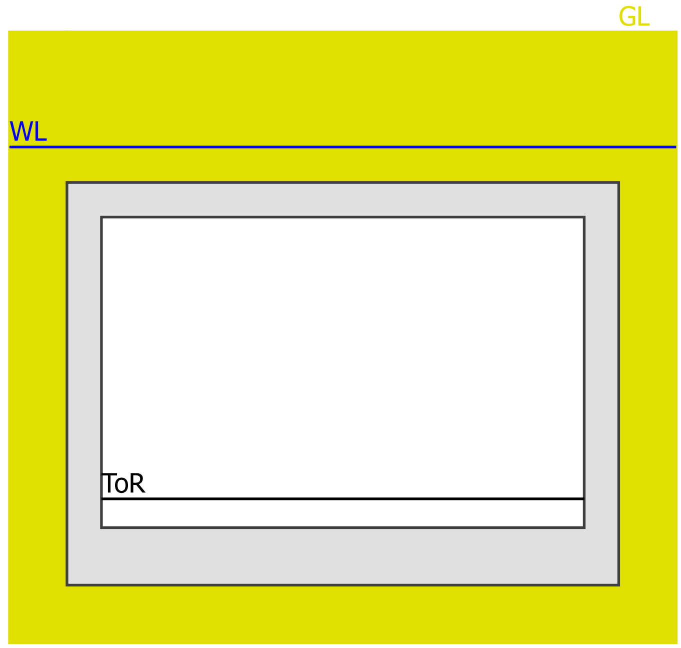
Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 34.341 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10934+ 0.000
Original Ground Level 289.000
Groundwater Level 279.000
Top of Rail 248.409
Top of Base 245.909
Founding Level 240.909



COVERED TROUGH
Trough Depth = 43.091 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 1882.000 psf
FoS Against Flotation = 0.767
Additional tie down force required 83.621 kips

WALL BASE SECTION
RC SECTION DESIGN
Bending Checks
Required capacity = 871.845 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 2.00 0.75 0.00
Smaller Bar 2.00 0.75 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 31.334 2.250
Area (sq-in) 7.167 0.000
Neutral Axis Depth = 10.539 in
Section is Tension controlled.
Reinforcement Strain 6.038e-03
Compression-block depth = 8.431 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 874.572 kip-ft/ft
Shear Checks
Required capacity = 156.151 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.589 sq in/ft run
Shear Capacity Provided = 166.396 kip

WALL PART-HEIGHT SECTION
RC SECTION DESIGN
Bending Checks
Required capacity = 435.922 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.50 0.00 0.00
Smaller Bar 1.50 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.000 2.250
Area (sq-in) 3.534 0.000
Neutral Axis Depth = 5.197 in
Section is Tension controlled.
Reinforcement Strain 1.547e-02
Compression-block depth = 4.158 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 475.873 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 37.987 kip

ROOF MIDSPAN SECTION
RC SECTION DESIGN
Bending Checks
Required capacity = 540.474 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.62 0.00 0.00
Smaller Bar 1.62 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 31.938 2.250
Area (sq-in) 4.148 0.000
Neutral Axis Depth = 6.100 in
Section is Tension controlled.
Reinforcement Strain 1.271e-02
Compression-block depth = 4.880 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 550.586 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 37.912 kip

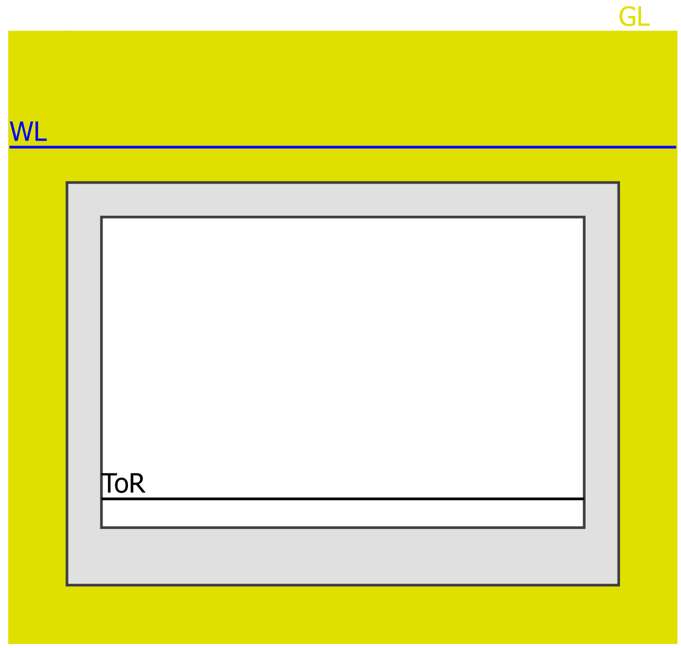
ROOF ROOT SECTION
RC SECTION DESIGN
Bending Checks
Required capacity = 1080.947 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 2.00 1.50 0.00
Smaller Bar 2.00 1.50 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 30.400 2.250
Area (sq-in) 9.817 0.000
Neutral Axis Depth = 14.437 in
Section is in transition to Compression Control.
Reinforcement Strain 3.597e-03
Compression-block depth = 11.550 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 1087.900 kip-ft/ft
Shear Checks
Required capacity = 144.126 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.589 sq in/ft run
Shear Capacity Provided = 161.437 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10934+ 2.000
Original Ground Level 289.000
Groundwater Level 279.000
Top of Rail 248.423
Top of Base 245.923
Founding Level 240.923



COVERED TROUGH
Trough Depth = 43.077 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 1882.000 psf
FoS Against Flotation = 0.767
Additional tie down force required 83.555 kips

WALL BASE SECTION
RC SECTION DESIGN
Bending Checks
Required capacity = 871.563 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	2.00	0.75	0.00
Smaller Bar	2.00	0.75	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.334	2.250
Area (sq-in)	7.167	0.000

Neutral Axis Depth = 10.539 in
Section is Tension controlled.
Reinforcement Strain 6.038e-03
Compression-block depth = 8.431 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 874.572 kip-ft/ft
Shear Checks
Required capacity = 156.101 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.589 sq in/ft run
Shear Capacity Provided = 166.396 kip

WALL PART-HEIGHT SECTION
RC SECTION DESIGN
Bending Checks
Required capacity = 435.782 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.50	0.00	0.00
Smaller Bar	1.50	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.000	2.250
Area (sq-in)	3.534	0.000

Neutral Axis Depth = 5.197 in
Section is Tension controlled.
Reinforcement Strain 1.547e-02
Compression-block depth = 4.158 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 475.873 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 37.987 kip

ROOF MIDSPAN SECTION
RC SECTION DESIGN
Bending Checks
Required capacity = 540.248 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.62	0.00	0.00
Smaller Bar	1.62	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.938	2.250
Area (sq-in)	4.148	0.000

Neutral Axis Depth = 6.100 in
Section is Tension controlled.
Reinforcement Strain 1.271e-02
Compression-block depth = 4.880 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 550.586 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 37.912 kip

ROOF ROOT SECTION
RC SECTION DESIGN
Bending Checks
Required capacity = 1080.495 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	2.00	1.50	0.00
Smaller Bar	2.00	1.50	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	30.400	2.250
Area (sq-in)	9.817	0.000

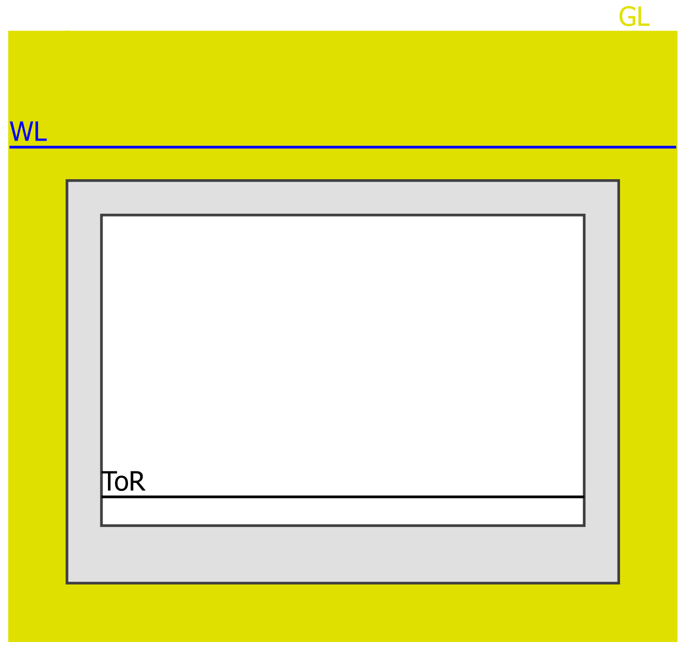
Neutral Axis Depth = 14.437 in
Section is in transition to Compression Control.
Reinforcement Strain 3.597e-03
Compression-block depth = 11.550 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 1087.900 kip-ft/ft
Shear Checks
Required capacity = 144.066 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.589 sq in/ft run
Shear Capacity Provided = 161.437 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10934+25.000
Original Ground Level 289.000
Groundwater Level 279.000
Top of Rail 248.597
Top of Base 246.097
Founding Level 241.097



COVERED TROUGH
Trough Depth =   42.903 ft
Base Thickness =   5.000 ft
Min Wall Thickness =   3.000 ft
Trough internal width =   42.000 ft
Design Load factors
DL   EH   WA   LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 1882.000 psf
FoS Against Flotation = 0.771
Additional tie down force required   82.776 kips

WALL BASE SECTION
RC SECTION DESIGN
Bending Checks
Required capacity = 868.267 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	2.00	0.75	0.00
Smaller Bar	2.00	0.75	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.334	2.250
Area (sq-in)	7.167	0.000

Neutral Axis Depth =   10.539 in
Section is Tension controlled.
Reinforcement Strain 6.038e-03
Compression-block depth =   8.431 in
Resistance factor (Phi) =   0.90
Moment Capacity (Phi.Mn) = 874.572 kip-ft/ft
Shear Checks
Required capacity = 155.511 kip
Shear Link Spacing L =   2.000 in
Shear Link Spacing T =   6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area =   0.589 sq in/ft run
Shear Capacity Provided = 166.396 kip

WALL PART-HEIGHT SECTION
RC SECTION DESIGN
Bending Checks
Required capacity = 434.134 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.50	0.00	0.00
Smaller Bar	1.50	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.000	2.250
Area (sq-in)	3.534	0.000

Neutral Axis Depth =   5.197 in
Section is Tension controlled.
Reinforcement Strain 1.547e-02
Compression-block depth =   4.158 in
Resistance factor (Phi) =   0.90
Moment Capacity (Phi.Mn) = 475.873 kip-ft/ft
Shear Checks
Required capacity =   0.000 kip
Shear Link Spacing L =   6.000 in
Shear Link Spacing T =   6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area =   0.000 sq in/ft run
Shear Capacity Provided =   37.987 kip

ROOF MIDSPAN SECTION
RC SECTION DESIGN
Bending Checks
Required capacity = 537.603 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.62	0.00	0.00
Smaller Bar	1.62	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.938	2.250
Area (sq-in)	4.148	0.000

Neutral Axis Depth =   6.100 in
Section is Tension controlled.
Reinforcement Strain 1.271e-02
Compression-block depth =   4.880 in
Resistance factor (Phi) =   0.90
Moment Capacity (Phi.Mn) = 550.586 kip-ft/ft
Shear Checks
Required capacity =   0.000 kip
Shear Link Spacing L =   6.000 in
Shear Link Spacing T =   6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area =   0.000 sq in/ft run
Shear Capacity Provided =   37.912 kip

ROOF ROOT SECTION
RC SECTION DESIGN
Bending Checks
Required capacity = 1075.206 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	2.00	1.50	0.00
Smaller Bar	2.00	1.50	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	30.400	2.250
Area (sq-in)	9.817	0.000

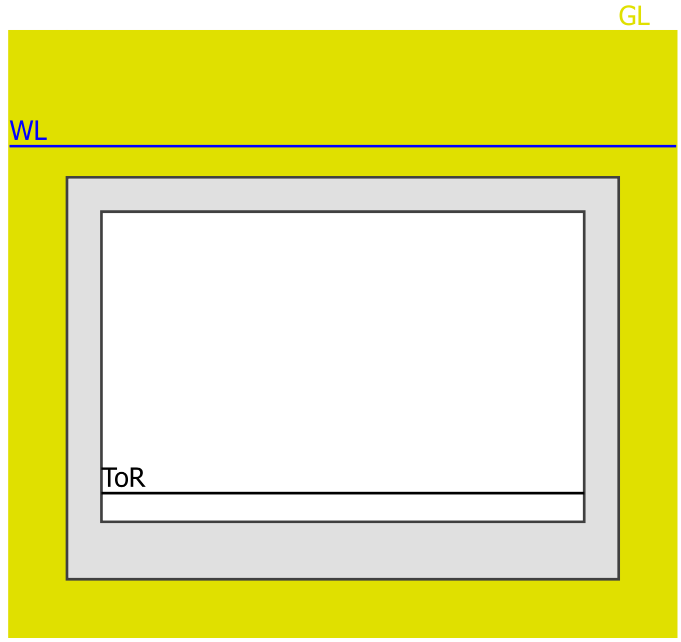
Neutral Axis Depth =   14.437 in
Section is in transition to Compression Control.
Reinforcement Strain 3.597e-03
Compression-block depth =   11.550 in
Resistance factor (Phi) =   0.90
Moment Capacity (Phi.Mn) = 1087.900 kip-ft/ft
Shear Checks
Required capacity = 143.361 kip
Shear Link Spacing L =   2.000 in
Shear Link Spacing T =   6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area =   0.589 sq in/ft run
Shear Capacity Provided = 161.437 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10934+50.000
Original Ground Level 289.000
Groundwater Level 279.000
Top of Rail 248.791
Top of Base 246.291
Founding Level 241.291



COVERED TROUGH
Trough Depth = 42.709 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 1882.000 psf
FoS Against Flotation = 0.775
Additional tie down force required 81.902 kips

WALL BASE SECTION
RC SECTION DESIGN
Bending Checks
Required capacity = 864.565 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	2.00	0.75	0.00
Smaller Bar	2.00	0.75	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.334	2.250
Area (sq-in)	7.167	0.000

Neutral Axis Depth = 10.539 in
Section is Tension controlled.
Reinforcement Strain 6.038e-03
Compression-block depth = 8.431 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 874.572 kip-ft/ft
Shear Checks
Required capacity = 154.848 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.589 sq in/ft run
Shear Capacity Provided = 166.396 kip

WALL PART-HEIGHT SECTION
RC SECTION DESIGN
Bending Checks
Required capacity = 432.283 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.50	0.00	0.00
Smaller Bar	1.50	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.000	2.250
Area (sq-in)	3.534	0.000

Neutral Axis Depth = 5.197 in
Section is Tension controlled.
Reinforcement Strain 1.547e-02
Compression-block depth = 4.158 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 475.873 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 37.987 kip

ROOF MIDSPAN SECTION
RC SECTION DESIGN
Bending Checks
Required capacity = 534.633 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.62	0.00	0.00
Smaller Bar	1.62	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.938	2.250
Area (sq-in)	4.148	0.000

Neutral Axis Depth = 6.100 in
Section is Tension controlled.
Reinforcement Strain 1.271e-02
Compression-block depth = 4.880 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 550.586 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 37.912 kip

ROOF ROOT SECTION
RC SECTION DESIGN
Bending Checks
Required capacity = 1069.266 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	2.00	1.50	0.00
Smaller Bar	2.00	1.50	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	30.400	2.250
Area (sq-in)	9.817	0.000

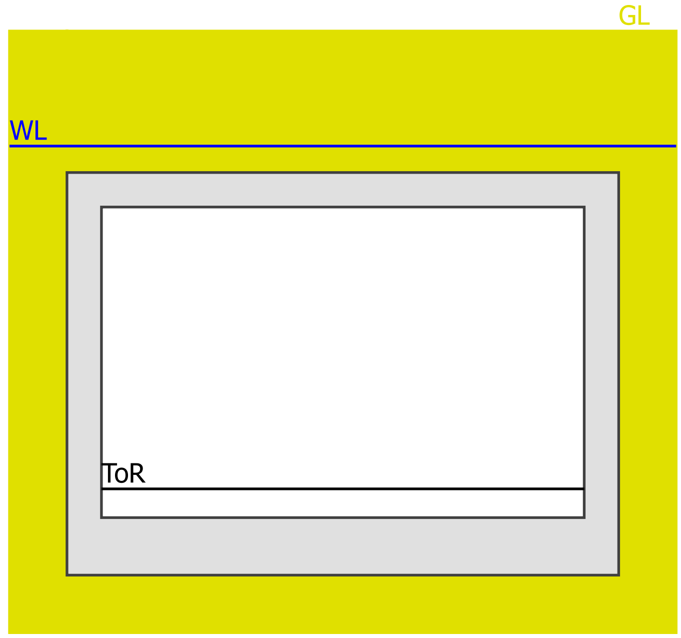
Neutral Axis Depth = 14.437 in
Section is in transition to Compression Control.
Reinforcement Strain 3.597e-03
Compression-block depth = 11.550 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 1087.900 kip-ft/ft
Shear Checks
Required capacity = 142.569 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.589 sq in/ft run
Shear Capacity Provided = 161.437 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10935+ 0.000
Original Ground Level 289.000
Groundwater Level 279.000
Top of Rail 249.200
Top of Base 246.700
Founding Level 241.700



COVERED TROUGH
Trough Depth = 42.300 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 1882.000 psf
FoS Against Flotation = 0.783
Additional tie down force required 80.065 kips

WALL BASE SECTION
RC SECTION DESIGN
Bending Checks
Required capacity = 856.789 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	2.00	0.75	0.00
Smaller Bar	2.00	0.75	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.334	2.250
Area (sq-in)	7.167	0.000

Neutral Axis Depth = 10.539 in
Section is Tension controlled.
Reinforcement Strain 6.038e-03
Compression-block depth = 8.431 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 874.572 kip-ft/ft
Shear Checks
Required capacity = 153.455 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.589 sq in/ft run
Shear Capacity Provided = 166.396 kip

WALL PART-HEIGHT SECTION
RC SECTION DESIGN
Bending Checks
Required capacity = 428.394 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.50	0.00	0.00
Smaller Bar	1.50	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.000	2.250
Area (sq-in)	3.534	0.000

Neutral Axis Depth = 5.197 in
Section is Tension controlled.
Reinforcement Strain 1.547e-02
Compression-block depth = 4.158 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 475.873 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 37.987 kip

ROOF MIDSPAN SECTION
RC SECTION DESIGN
Bending Checks
Required capacity = 528.393 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.62	0.00	0.00
Smaller Bar	1.62	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.938	2.250
Area (sq-in)	4.148	0.000

Neutral Axis Depth = 6.100 in
Section is Tension controlled.
Reinforcement Strain 1.271e-02
Compression-block depth = 4.880 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 550.586 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 37.912 kip

ROOF ROOT SECTION
RC SECTION DESIGN
Bending Checks
Required capacity = 1056.787 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	2.00	1.50	0.00
Smaller Bar	2.00	1.50	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	30.400	2.250
Area (sq-in)	9.817	0.000

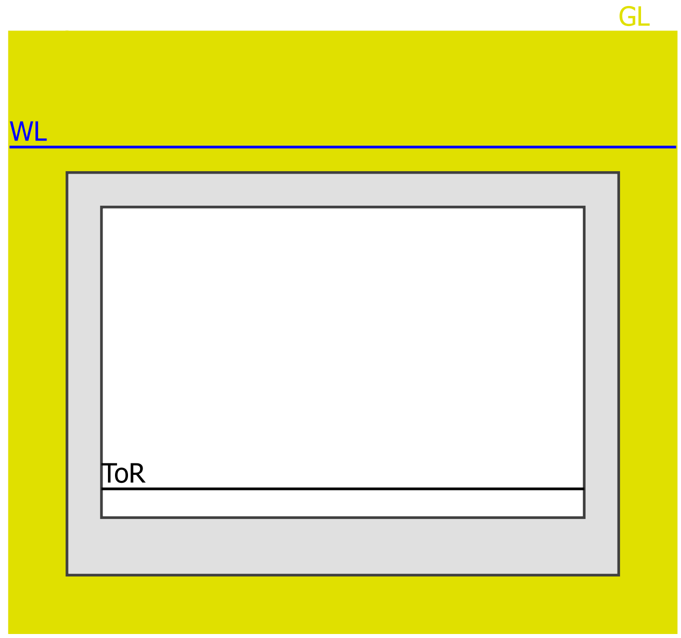
Neutral Axis Depth = 14.437 in
Section is in transition to Compression Control.
Reinforcement Strain 3.597e-03
Compression-block depth = 11.550 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 1087.900 kip-ft/ft
Shear Checks
Required capacity = 140.905 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.589 sq in/ft run
Shear Capacity Provided = 161.437 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10935+10.000
Original Ground Level 289.000
Groundwater Level 279.000
Top of Rail 249.285
Top of Base 246.785
Founding Level 241.785



COVERED TROUGH
Trough Depth = 42.215 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 1882.000 psf
FoS Against Flotation = 0.785
Additional tie down force required 79.683 kips

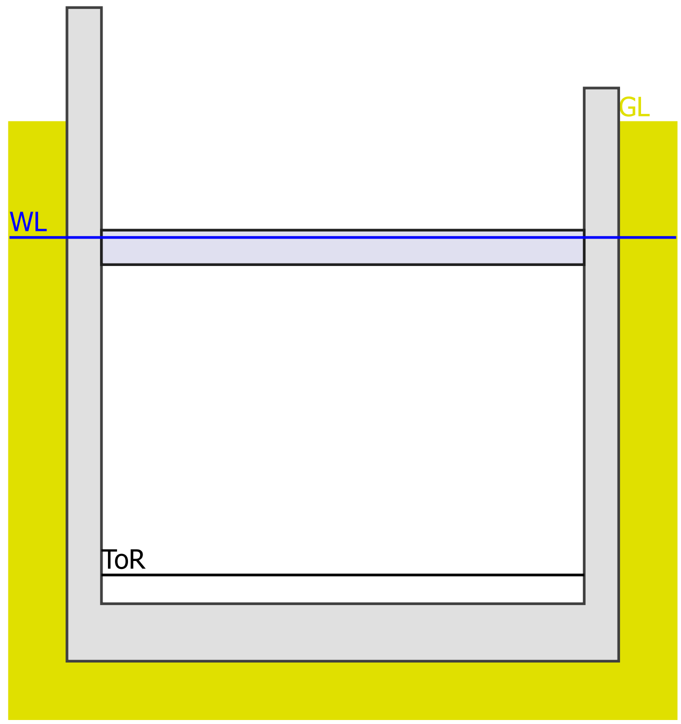
WALL BASE SECTION
RC SECTION DESIGN
Bending Checks
Required capacity = 855.174 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 2.00 0.75 0.00
Smaller Bar 2.00 0.75 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 31.334 2.250
Area (sq-in) 7.167 0.000
Neutral Axis Depth = 10.539 in
Section is Tension controlled.
Reinforcement Strain 6.038e-03
Compression-block depth = 8.431 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 874.572 kip-ft/ft
Shear Checks
Required capacity = 153.165 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.589 sq in/ft run
Shear Capacity Provided = 166.396 kip

WALL PART-HEIGHT SECTION
RC SECTION DESIGN
Bending Checks
Required capacity = 427.587 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.50 0.00 0.00
Smaller Bar 1.50 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.000 2.250
Area (sq-in) 3.534 0.000
Neutral Axis Depth = 5.197 in
Section is Tension controlled.
Reinforcement Strain 1.547e-02
Compression-block depth = 4.158 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 475.873 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 37.987 kip

ROOF MIDSPAN SECTION
RC SECTION DESIGN
Bending Checks
Required capacity = 527.098 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.62 0.00 0.00
Smaller Bar 1.62 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 31.938 2.250
Area (sq-in) 4.148 0.000
Neutral Axis Depth = 6.100 in
Section is Tension controlled.
Reinforcement Strain 1.271e-02
Compression-block depth = 4.880 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 550.586 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 37.912 kip

ROOF ROOT SECTION
RC SECTION DESIGN
Bending Checks
Required capacity = 1054.195 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 2.00 1.50 0.00
Smaller Bar 2.00 1.50 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 30.400 2.250
Area (sq-in) 9.817 0.000
Neutral Axis Depth = 14.437 in
Section is in transition to Compression Control.
Reinforcement Strain 3.597e-03
Compression-block depth = 11.550 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 1087.900 kip-ft/ft
Shear Checks
Required capacity = 140.559 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.589 sq in/ft run
Shear Capacity Provided = 161.437 kip

Section at 10935+50.000
Original Ground Level 289.000
Groundwater Level 279.000
Top of Rail 249.635
Top of Base 247.135
Founding Level 242.135



BRACED U-TROUGH
Trough Depth = 41.865 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 420.000 psf
FoS Against Flotation = 0.749
Additional tie down force required 82.917 kips

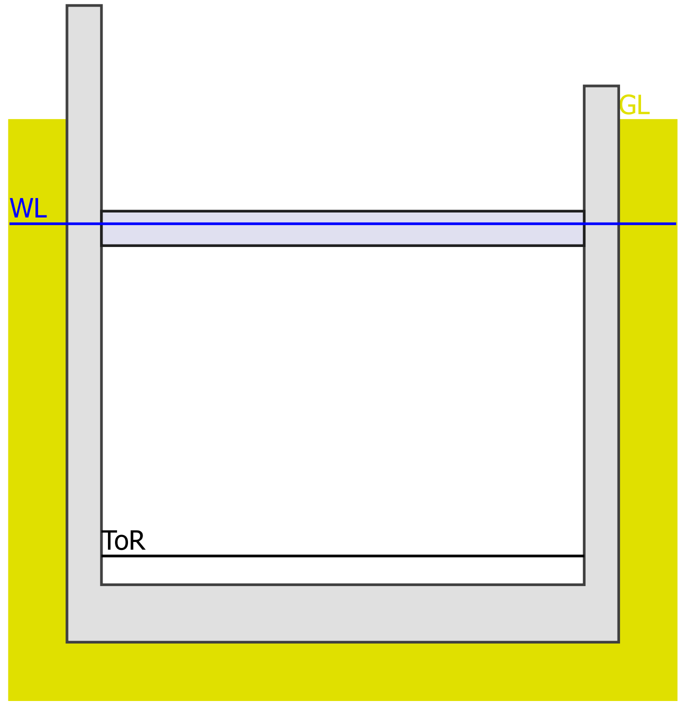
BASE OF WALL AT 41.865 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 264.202 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.12 0.00 0.00
Smaller Bar 1.12 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.188 2.250
Area (sq-in) 1.988 0.000
Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 50.746 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.589 sq in/ft run
Shear Capacity Provided = 170.929 kip

WALL AT 16.519 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 95.421 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.00 0.00 0.00
Smaller Bar 1.00 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.250 2.250
Area (sq-in) 1.571 0.000
Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL PROP
Prop Spacing = 10.000 ft
Prop Force = 35.190 kip/ft
Force per prop = 351.903 kips

WALL AT PROP LEVEL (10.865 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 282.819 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.25 0.00 0.00
Smaller Bar 1.25 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.125 2.250
Area (sq-in) 2.454 0.000
Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 33.394 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

Section at 10936+ 0.000
Original Ground Level 287.990
Groundwater Level 279.000
Top of Rail 250.096
Top of Base 247.596
Founding Level 242.596



BRACED U-TROUGH
Trough Depth = 40.394 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 420.000 psf
FoS Against Flotation = 0.746
Additional tie down force required 82.222 kips

BASE OF WALL AT 40.394 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 267.051 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.188	2.250
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 51.022 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.589 sq in/ft run
Shear Capacity Provided = 170.929 kip

WALL AT 17.240 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 106.306 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL PROP
Prop Spacing = 10.000 ft
Prop Force = 36.322 kip/ft
Force per prop = 363.220 kips

WALL AT PROP LEVEL (9.394 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 277.119 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.188	2.250
Area (sq-in)	1.988	0.000

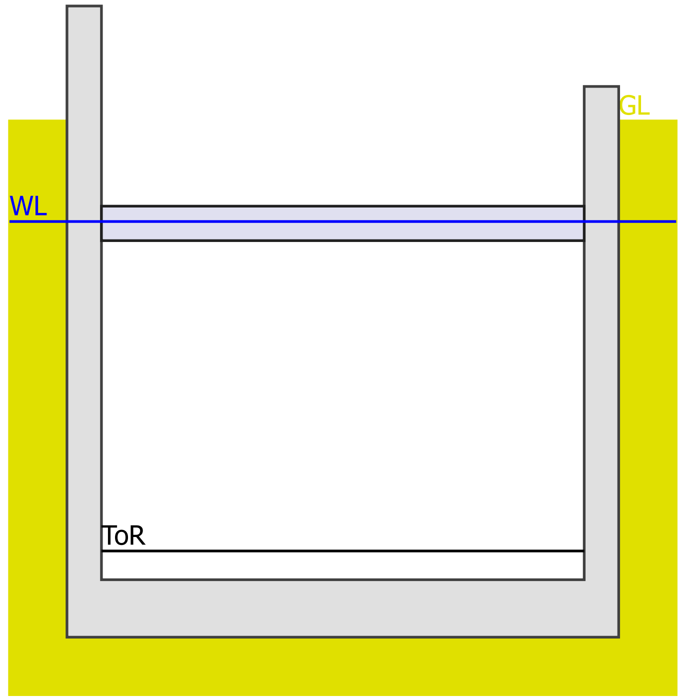
Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 32.555 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.209 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10936+25.000
Original Ground Level 287.750
Groundwater Level 279.000
Top of Rail 250.337
Top of Base 247.837
Founding Level 242.837



BRACED U-TROUGH
Trough Depth = 39.913 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 420.000 psf
FoS Against Flotation = 0.747
Additional tie down force required 81.592 kips

BASE OF WALL AT 39.913 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 267.892 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.188	2.250
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 51.103 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.589 sq in/ft run
Shear Capacity Provided = 170.929 kip

WALL AT 17.453 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 109.555 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL PROP
Prop Spacing = 10.000 ft
Prop Force = 36.796 kip/ft
Force per prop = 367.963 kips

WALL AT PROP LEVEL (8.913 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 275.438 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.188	2.250
Area (sq-in)	1.988	0.000

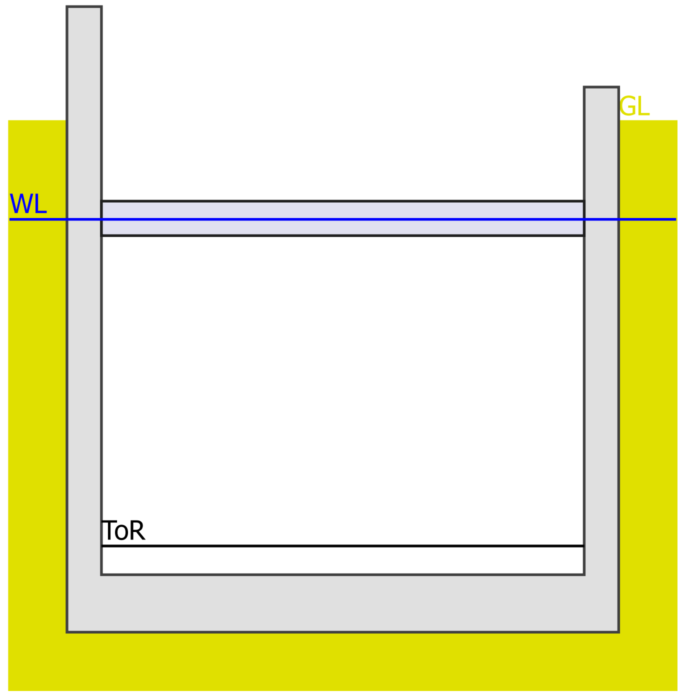
Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 32.290 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.209 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10936+50.000
Original Ground Level 287.510
Groundwater Level 279.000
Top of Rail 250.583
Top of Base 248.083
Founding Level 243.083



BRACED U-TROUGH
Trough Depth = 39.427 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 420.000 psf
FoS Against Flotation = 0.748
Additional tie down force required 80.939 kips

BASE OF WALL AT 39.427 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 267.541 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.188	2.250
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 50.994 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.589 sq in/ft run
Shear Capacity Provided = 170.929 kip

WALL AT 17.657 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 112.692 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL PROP
Prop Spacing = 10.000 ft
Prop Force = 37.340 kip/ft
Force per prop = 373.399 kips

WALL AT PROP LEVEL (8.427 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 273.823 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

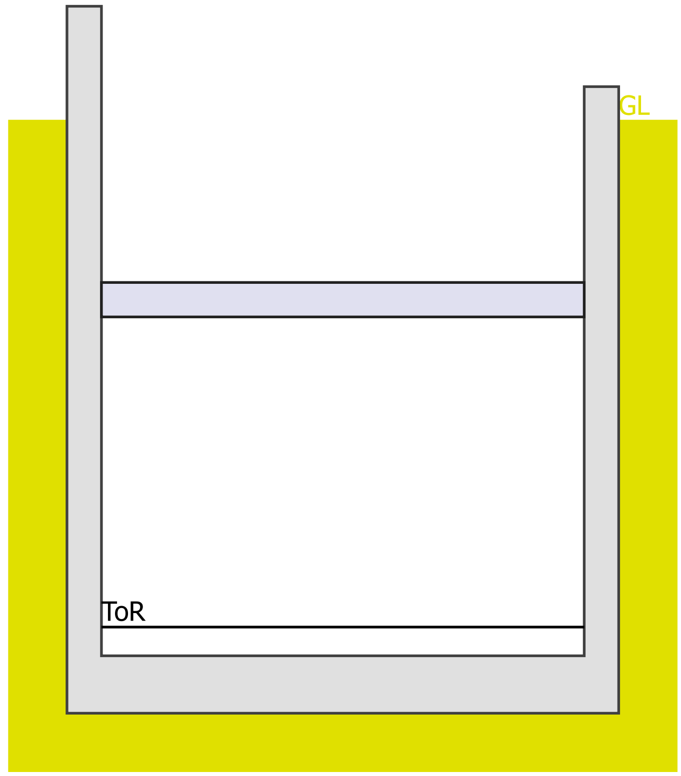
Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.188	2.250
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 31.914 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.209 kip

Section at 10939+75.000
Original Ground Level 298.435
Groundwater Level 235.000
Top of Rail 254.388
Top of Base 251.888
Founding Level 246.888



BRACED U-TROUGH
Trough Depth = 46.547 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf

BASE OF WALL AT 46.547 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 232.315 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.188	2.250
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 22.516 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.209 kip

WALL AT 13.527 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 57.587 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL PROP
Prop Spacing = 10.000 ft
Prop Force = 38.429 kip/ft
Force per prop = 384.293 kips

WALL AT PROP LEVEL (15.547 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 326.281 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

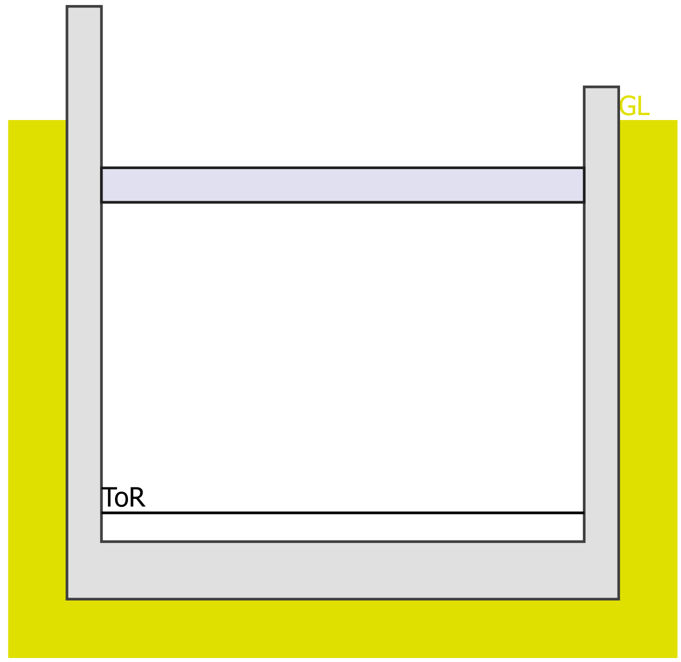
Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.125	2.250
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 31.991 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

Fresno Grade Separation
Preliminary Design

Section at 10940+ 0.000
Original Ground Level 288.760
Groundwater Level 235.000
Top of Rail 254.726
Top of Base 252.226
Founding Level 247.226



BRACED U-TROUGH
Trough Depth = 36.534 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf

BASE OF WALL AT 36.534 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 212.730 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 25.859 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL AT 18.634 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 137.936 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL PROP
Prop Spacing = 10.000 ft
Prop Force = 44.949 kip/ft
Force per prop = 449.487 kips

WALL AT PROP LEVEL (5.534 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 268.611 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

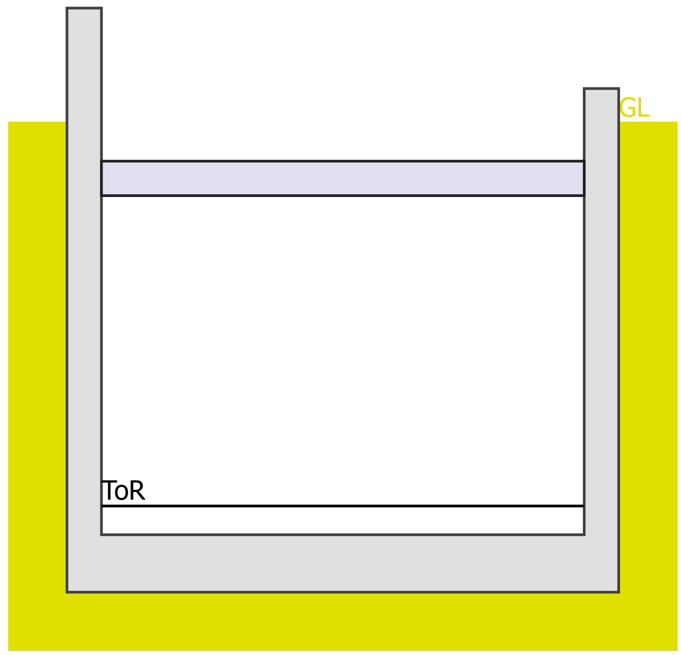
Layer	Tension	Compression
Depth (in)	32.188	2.250
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 23.844 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.209 kip

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation Preliminary Design

Section at 10940+50.000
Original Ground Level 288.740
Groundwater Level 235.000
Top of Rail 255.422
Top of Base 252.922
Founding Level 247.922



BRACED U-TROUGH
Trough Depth = 35.818 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf

BASE OF WALL AT 35.818 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 232.195 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.188	2.250
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 27.111 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.209 kip

WALL AT 18.813 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 141.018 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL AT PROP LEVEL (4.818 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 266.585 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

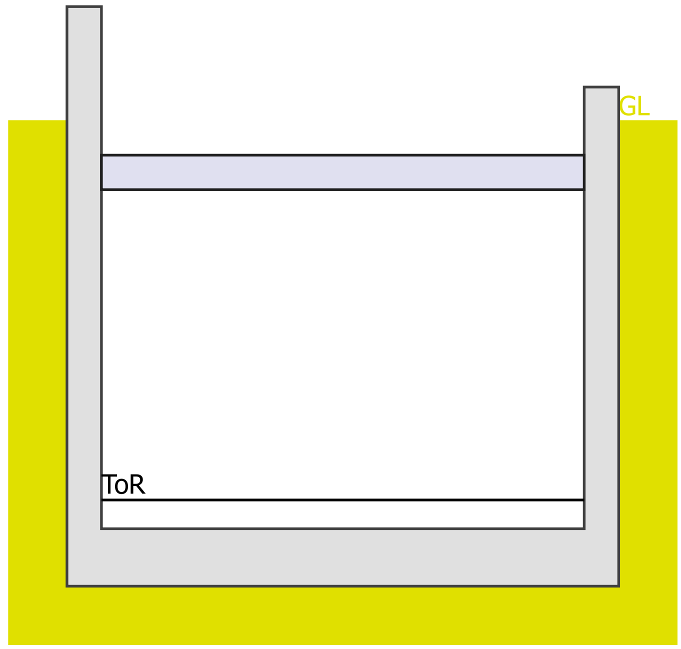
Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.188	2.250
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 23.614 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.209 kip

WALL PROP
Prop Spacing = 10.000 ft
Prop Force = 46.804 kip/ft
Force per prop = 468.038 kips

Section at 10941+ 0.000
Original Ground Level 289.070
Groundwater Level 235.000
Top of Rail 256.145
Top of Base 253.645
Founding Level 248.645



BRACED U-TROUGH
Trough Depth = 35.425 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf

BASE OF WALL AT 35.425 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 255.107 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.188	2.250
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 28.381 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.209 kip

WALL AT 18.901 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 142.532 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL PROP
Prop Spacing = 10.000 ft
Prop Force = 48.006 kip/ft
Force per prop = 480.057 kips

WALL AT PROP LEVEL (4.425 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 265.594 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

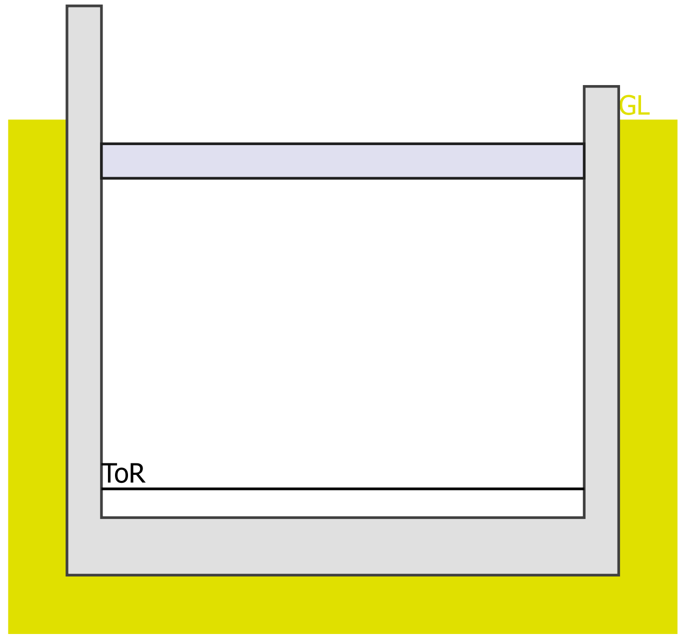
Layer	Tension	Compression
Depth (in)	32.188	2.250
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 23.656 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.209 kip

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation Preliminary Design

Section at 10941+50.000
Original Ground Level 288.880
Groundwater Level 235.000
Top of Rail 256.893
Top of Base 254.393
Founding Level 249.393



BRACED U-TROUGH
Trough Depth = 34.487 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf

BASE OF WALL AT 34.487 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 280.822 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.125	2.250
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 29.768 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

WALL AT 19.081 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 145.644 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL PROP
Prop Spacing = 20.000 ft
Prop Force = 51.570 kip/ft
Force per prop = 1031.406 kips

WALL AT PROP LEVEL (3.487 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 263.565 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.188	2.250
Area (sq-in)	1.988	0.000

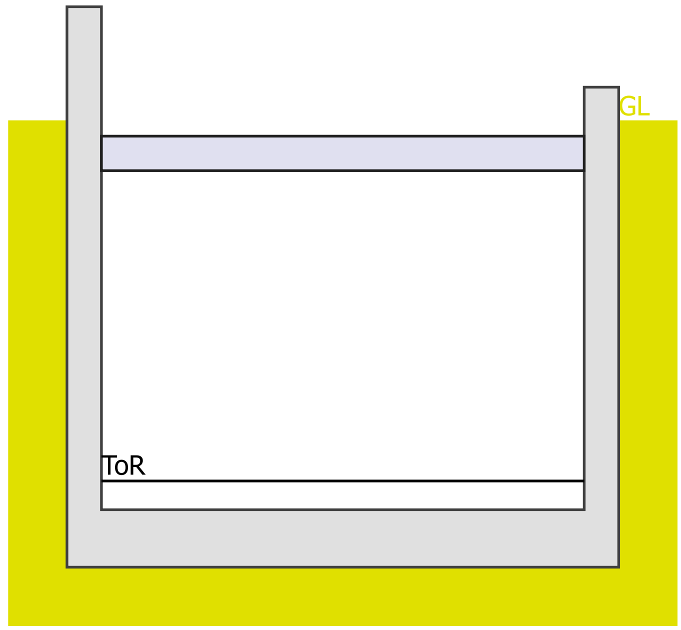
Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 23.403 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.209 kip

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10942+ 0.000
Original Ground Level 288.920
Groundwater Level 235.000
Top of Rail 257.660
Top of Base 255.160
Founding Level 250.160

BRACED U-TROUGH
Trough Depth = 33.760 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf



BASE OF WALL AT 33.760 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 310.357 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.125	2.250
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 31.166 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

WALL AT 19.191 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 147.555 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.00	0.00	0.00
Smaller Bar	1.00	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.250	2.250
Area (sq-in)	1.571	0.000

Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL PROP
Prop Spacing = 20.000 ft
Prop Force = 55.223 kip/ft
Force per prop = 1104.465 kips

WALL AT PROP LEVEL (2.760 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 262.325 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

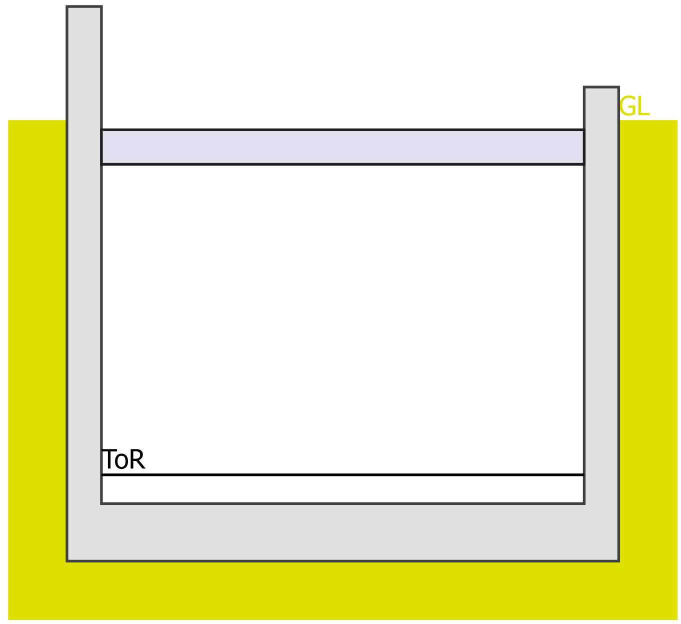
Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.188	2.250
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 23.359 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.209 kip

Fresno Grade Separation
Preliminary Design

Section at 10942+50.000
Original Ground Level 289.150
Groundwater Level 235.000
Top of Rail 258.435
Top of Base 255.935
Founding Level 250.935



BRACED U-TROUGH
Trough Depth = 33.215 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf

BASE OF WALL AT 33.215 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 343.519 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.38 0.00 0.00
Smaller Bar 1.38 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.062 2.250
Area (sq-in) 2.970 0.000
Neutral Axis Depth = 4.367 in
Section is Tension controlled.
Reinforcement Strain 1.902e-02
Compression-block depth = 3.494 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 405.138 kip-ft/ft
Shear Checks
Required capacity = 32.572 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.061 kip

WALL AT 19.256 ABOVE BASE
RC SECTION DESIGN
Bending Checks
Required capacity = 148.700 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.00 0.00 0.00
Smaller Bar 1.00 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.250 2.250
Area (sq-in) 1.571 0.000
Neutral Axis Depth = 2.310 in
Section is Tension controlled.
Reinforcement Strain 3.888e-02
Compression-block depth = 1.848 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 221.430 kip-ft/ft
Shear Checks
Required capacity = 0.000 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.283 kip

WALL PROP
Prop Spacing = 20.000 ft
Prop Force = 58.680 kip/ft
Force per prop = 1173.590 kips

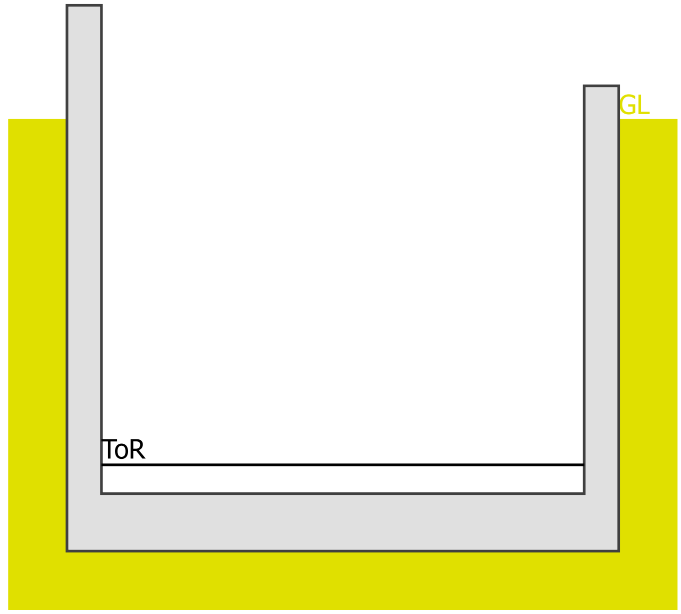
WALL AT PROP LEVEL (2.215 BELOW GROUND)
RC SECTION DESIGN
Bending Checks
Required capacity = 261.584 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.12 0.00 0.00
Smaller Bar 1.12 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.188 2.250
Area (sq-in) 1.988 0.000
Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 23.497 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.209 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation Preliminary Design

Section at 10943+ 0.000
Original Ground Level 289.160
Groundwater Level 235.000
Top of Rail 259.210
Top of Base 256.710
Founding Level 251.710



UN-BRACED U-TROUGH
Trough Depth = 32.450 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf

WALL ROOT SECTION at 32.450 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 1400.272 kip-ft
Section thickness = 36.000 in
Bar Spacing = 4.00 in

Layer	T1	T2	C1
Larger Bar	2.00	2.00	1.75
Smaller Bar	2.00	2.00	1.75

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	29.500	3.375
Area (sq-in)	18.850	7.216

Neutral Axis Depth = 17.108 in
Section is in transition to Compression Control.
Reinforcement Strain 2.524e-03
Compression-block depth = 13.687 in
Resistance factor (Phi) = 0.75
Moment Capacity (Phi.Mn) = 1695.355 kip-ft/ft
Shear Checks
Required capacity = 652.396 kip
Shear Link Spacing L = 3.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 3.534 sq in/ft run
Shear Capacity Provided = 771.332 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 335.655 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.875	2.500
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 541.424 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip

SECTION AT 20.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 620.430 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.75	0.00	0.00
Smaller Bar	1.75	0.00	0.00

Cracking Moment = 178.707 kip-ft

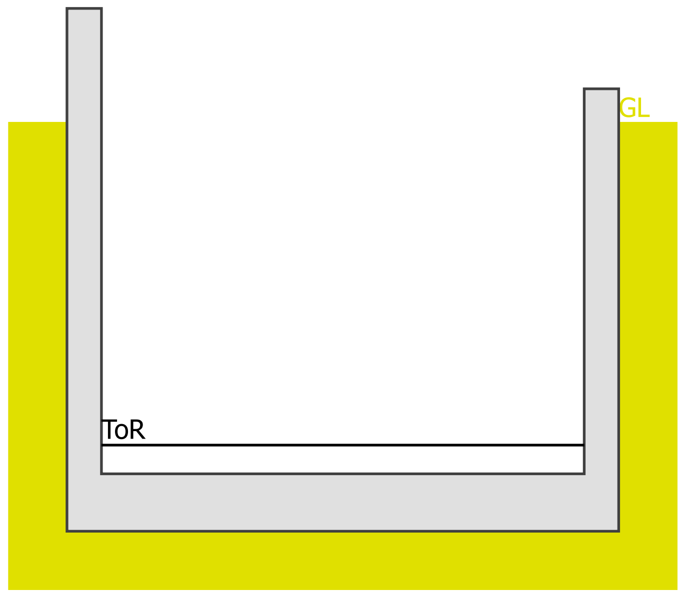
Layer	Tension	Compression
Depth (in)	31.625	2.500
Area (sq-in)	4.811	0.000

Neutral Axis Depth = 7.074 in
Section is Tension controlled.
Reinforcement Strain 1.041e-02
Compression-block depth = 5.659 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 623.346 kip-ft/ft
Shear Checks
Required capacity = 541.424 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 563.564 kip

Fresno Grade Separation
Preliminary Design

Section at 10943+50.000
Original Ground Level 288.010
Groundwater Level 235.000
Top of Rail 259.986
Top of Base 257.486
Founding Level 252.486

UN-BRACED U-TROUGH
Trough Depth = 30.524 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 30.524 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 1242.764 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	2.00	2.00	0.00
Smaller Bar	2.00	2.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	29.375	2.625
Area (sq-in)	12.566	0.000

Neutral Axis Depth = 18.480 in
Section is in transition to Compression Control.
Reinforcement Strain 2.093e-03
Compression-block depth = 14.784 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 1243.110 kip-ft/ft
Shear Checks
Required capacity = 639.410 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.625 in
Shear Reinforcement Area = 3.682 sq in/ft run
Shear Capacity Provided = 801.999 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 335.655 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.875	2.500
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 541.424 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip

SECTION AT 20.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 620.430 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.75	0.00	0.00
Smaller Bar	1.75	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.625	2.500
Area (sq-in)	4.811	0.000

Neutral Axis Depth = 7.074 in
Section is Tension controlled.
Reinforcement Strain 1.041e-02
Compression-block depth = 5.659 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 623.346 kip-ft/ft
Shear Checks
Required capacity = 541.424 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 563.564 kip

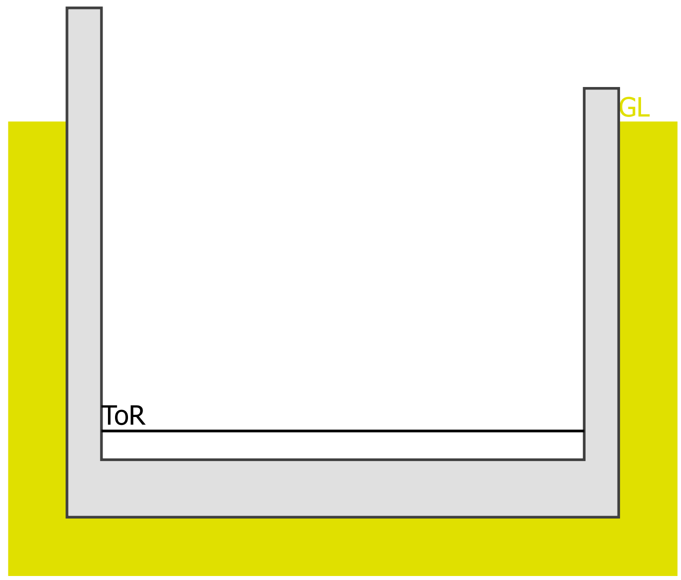
California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10944+ 0.000
Original Ground Level 287.570
Groundwater Level 235.000
Top of Rail 260.761
Top of Base 258.261
Founding Level 253.261

UN-BRACED U-TROUGH
Trough Depth = 29.309 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 29.309 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 1150.819 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 2.00 1.75 0.00
Smaller Bar 2.00 1.75 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 29.695 2.625
Area (sq-in) 11.094 0.000
Neutral Axis Depth = 16.314 in
Section is in transition to Compression Control.
Reinforcement Strain 2.769e-03
Compression-block depth = 13.051 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 1156.638 kip-ft/ft
Shear Checks
Required capacity = 631.549 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.625 in
Shear Reinforcement Area = 3.682 sq in/ft run
Shear Capacity Provided = 810.618 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 335.655 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.25 0.00 0.00
Smaller Bar 1.25 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 31.875 2.500
Area (sq-in) 2.454 0.000
Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 541.424 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip

SECTION AT 20.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 620.430 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.75 0.00 0.00
Smaller Bar 1.75 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 31.625 2.500
Area (sq-in) 4.811 0.000
Neutral Axis Depth = 7.074 in
Section is Tension controlled.
Reinforcement Strain 1.041e-02
Compression-block depth = 5.659 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 623.346 kip-ft/ft
Shear Checks
Required capacity = 541.424 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 563.564 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

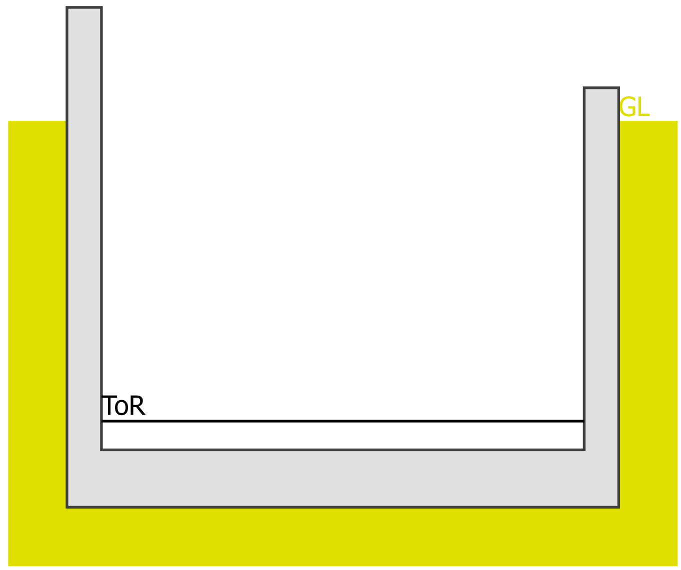
Fresno Grade Separation
Preliminary Design

Section at 10944+50.000
Original Ground Level 287.560
Groundwater Level 235.000
Top of Rail 261.536
Top of Base 259.036
Founding Level 254.036

UN-BRACED U-TROUGH
Trough Depth = 28.524 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 28.524 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 1094.395 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	2.00	1.75	0.00
Smaller Bar	2.00	1.75	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	29.695	2.625
Area (sq-in)	11.094	0.000

Neutral Axis Depth = 16.314 in
Section is in transition to Compression Control.
Reinforcement Strain 2.769e-03
Compression-block depth = 13.051 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 1156.638 kip-ft/ft
Shear Checks
Required capacity = 626.607 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.625 in
Shear Reinforcement Area = 3.682 sq in/ft run
Shear Capacity Provided = 810.618 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 335.655 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.875	2.500
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 541.424 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip

SECTION AT 20.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 620.430 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.75	0.00	0.00
Smaller Bar	1.75	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.625	2.500
Area (sq-in)	4.811	0.000

Neutral Axis Depth = 7.074 in
Section is Tension controlled.
Reinforcement Strain 1.041e-02
Compression-block depth = 5.659 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 623.346 kip-ft/ft
Shear Checks
Required capacity = 541.424 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 563.564 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

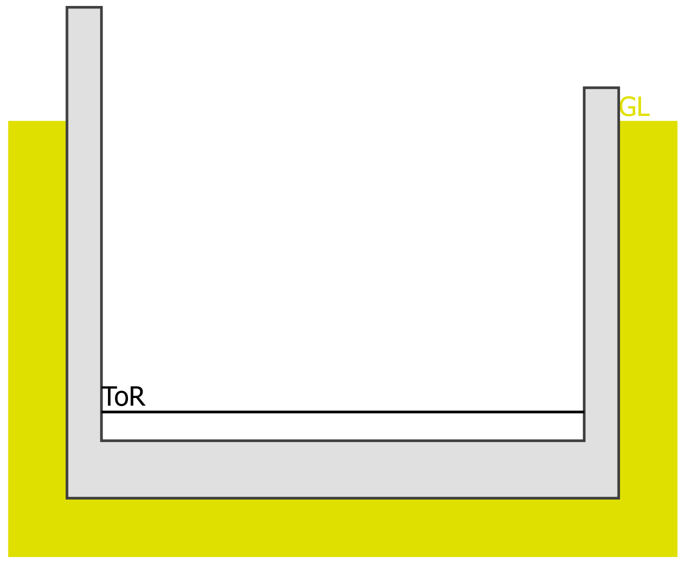
Fresno Grade Separation
Preliminary Design

Section at 10945+ 0.000
Original Ground Level 287.520
Groundwater Level 235.000
Top of Rail 262.311
Top of Base 259.811
Founding Level 254.811

UN-BRACED U-TROUGH
Trough Depth = 27.709 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 27.709 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 1038.242 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	2.00	1.50	0.00
Smaller Bar	2.00	1.50	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	30.025	2.625
Area (sq-in)	9.817	0.000

Neutral Axis Depth = 14.437 in
Section is in transition to Compression Control.
Reinforcement Strain 3.519e-03
Compression-block depth = 11.550 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 1071.333 kip-ft/ft
Shear Checks
Required capacity = 621.591 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.625 in
Shear Reinforcement Area = 3.682 sq in/ft run
Shear Capacity Provided = 819.522 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 335.655 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.875	2.500
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 541.424 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip

SECTION AT 20.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 620.430 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.75	0.00	0.00
Smaller Bar	1.75	0.00	0.00

Cracking Moment = 178.707 kip-ft

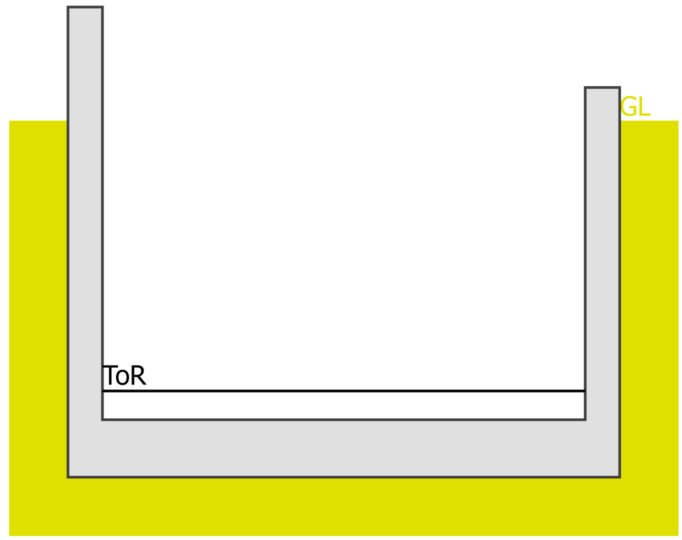
Layer	Tension	Compression
Depth (in)	31.625	2.500
Area (sq-in)	4.811	0.000

Neutral Axis Depth = 7.074 in
Section is Tension controlled.
Reinforcement Strain 1.041e-02
Compression-block depth = 5.659 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 623.346 kip-ft/ft
Shear Checks
Required capacity = 541.424 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 563.564 kip

Fresno Grade Separation
Preliminary Design

Section at 10945+50.000
Original Ground Level 286.490
Groundwater Level 235.000
Top of Rail 263.086
Top of Base 260.586
Founding Level 255.586

UN-BRACED U-TROUGH
Trough Depth = 25.904 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 25.904 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 922.450 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 2.00 1.25 0.00
Smaller Bar 2.00 1.25 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 30.357 2.625
Area (sq-in) 8.738 0.000
Neutral Axis Depth = 12.849 in
Section is in transition to Compression Control.
Reinforcement Strain 4.325e-03
Compression-block depth = 10.279 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 991.507 kip-ft/ft
Shear Checks
Required capacity = 610.898 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.625 in
Shear Reinforcement Area = 3.682 sq in/ft run
Shear Capacity Provided = 837.045 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 335.655 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.25 0.00 0.00
Smaller Bar 1.25 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 31.875 2.500
Area (sq-in) 2.454 0.000
Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 541.424 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip

SECTION AT 20.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 620.430 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.75 0.00 0.00
Smaller Bar 1.75 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 31.625 2.500
Area (sq-in) 4.811 0.000
Neutral Axis Depth = 7.074 in
Section is Tension controlled.
Reinforcement Strain 1.041e-02
Compression-block depth = 5.659 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 623.346 kip-ft/ft
Shear Checks
Required capacity = 541.424 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 563.564 kip

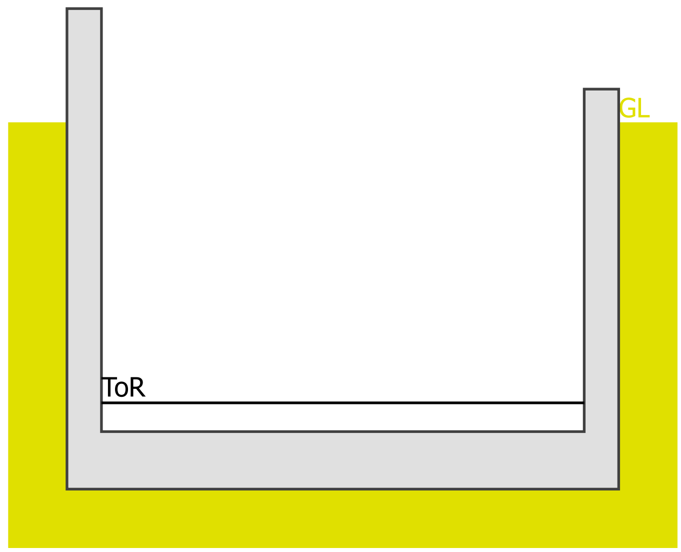
California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10946+ 0.000
Original Ground Level 288.130
Groundwater Level 235.000
Top of Rail 263.861
Top of Base 261.361
Founding Level 256.361

UN-BRACED U-TROUGH
Trough Depth = 26.769 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 26.769 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 976.477 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	2.00	1.25	0.00
Smaller Bar	2.00	1.25	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	30.357	2.625
Area (sq-in)	8.738	0.000

Neutral Axis Depth = 12.849 in
Section is in transition to Compression Control.
Reinforcement Strain 4.325e-03
Compression-block depth = 10.279 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 991.507 kip-ft/ft
Shear Checks
Required capacity = 615.950 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.625 in
Shear Reinforcement Area = 3.682 sq in/ft run
Shear Capacity Provided = 828.465 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 335.655 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.875	2.500
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 541.424 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip

SECTION AT 20.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 620.430 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.75	0.00	0.00
Smaller Bar	1.75	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.625	2.500
Area (sq-in)	4.811	0.000

Neutral Axis Depth = 7.074 in
Section is Tension controlled.
Reinforcement Strain 1.041e-02
Compression-block depth = 5.659 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 623.346 kip-ft/ft
Shear Checks
Required capacity = 541.424 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 563.564 kip

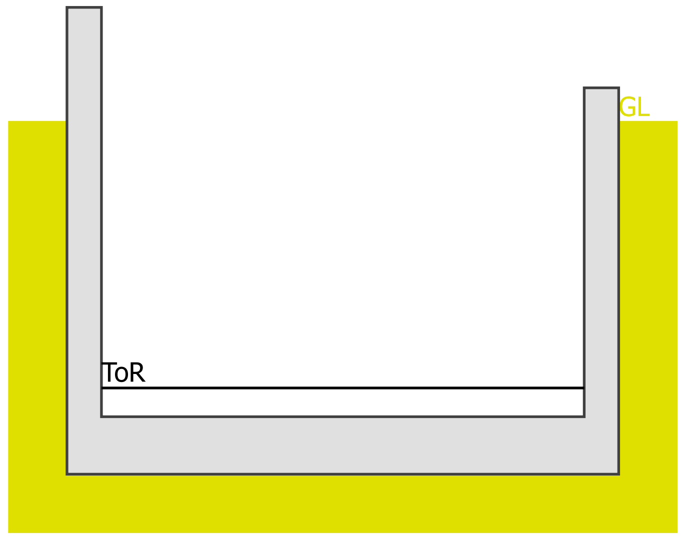
California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10946+50.000
Original Ground Level 287.750
Groundwater Level 235.000
Top of Rail 264.636
Top of Base 262.136
Founding Level 257.136

UN-BRACED U-TROUGH
Trough Depth = 25.614 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 25.614 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 904.905 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 2.00 1.00 0.00
Smaller Bar 2.00 1.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 30.675 2.625
Area (sq-in) 7.854 0.000
Neutral Axis Depth = 11.550 in
Section is Tension controlled.
Reinforcement Strain 5.149e-03
Compression-block depth = 9.240 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 920.860 kip-ft/ft
Shear Checks
Required capacity = 609.232 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.625 in
Shear Reinforcement Area = 3.682 sq in/ft run
Shear Capacity Provided = 837.045 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 335.655 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.25 0.00 0.00
Smaller Bar 1.25 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 31.875 2.500
Area (sq-in) 2.454 0.000
Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 541.424 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip

SECTION AT 20.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 620.430 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.75 0.00 0.00
Smaller Bar 1.75 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 31.625 2.500
Area (sq-in) 4.811 0.000
Neutral Axis Depth = 7.074 in
Section is Tension controlled.
Reinforcement Strain 1.041e-02
Compression-block depth = 5.659 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 623.346 kip-ft/ft
Shear Checks
Required capacity = 541.424 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 563.564 kip

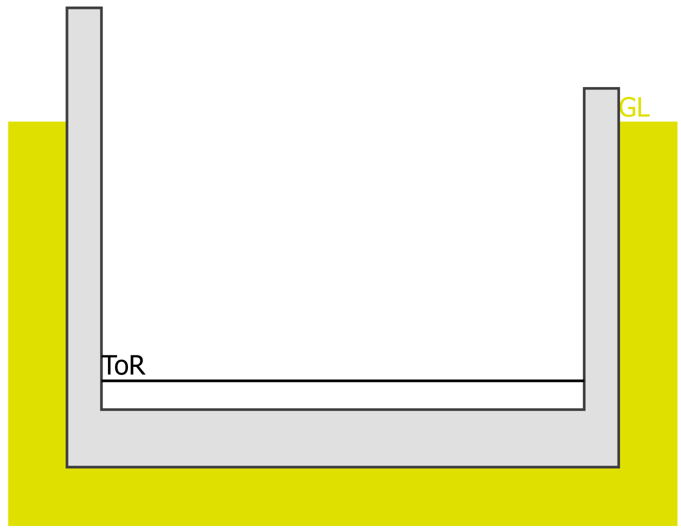
California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10947+ 0.000
Original Ground Level 287.830
Groundwater Level 235.000
Top of Rail 265.411
Top of Base 262.911
Founding Level 257.911

UN-BRACED U-TROUGH
Trough Depth = 24.919 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 24.919 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 864.072 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 2.00 1.00 0.00
Smaller Bar 2.00 1.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 30.675 2.625
Area (sq-in) 7.854 0.000
Neutral Axis Depth = 11.550 in
Section is Tension controlled.
Reinforcement Strain 5.149e-03
Compression-block depth = 9.240 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 920.860 kip-ft/ft
Shear Checks
Required capacity = 605.303 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.625 in
Shear Reinforcement Area = 3.682 sq in/ft run
Shear Capacity Provided = 844.698 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 335.655 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.25 0.00 0.00
Smaller Bar 1.25 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 31.875 2.500
Area (sq-in) 2.454 0.000
Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 541.424 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip

SECTION AT 20.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 620.430 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.75 0.00 0.00
Smaller Bar 1.75 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 31.625 2.500
Area (sq-in) 4.811 0.000
Neutral Axis Depth = 7.074 in
Section is Tension controlled.
Reinforcement Strain 1.041e-02
Compression-block depth = 5.659 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 623.346 kip-ft/ft
Shear Checks
Required capacity = 541.424 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 563.564 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08

Designed by : AJA

Checked by : YR/SS

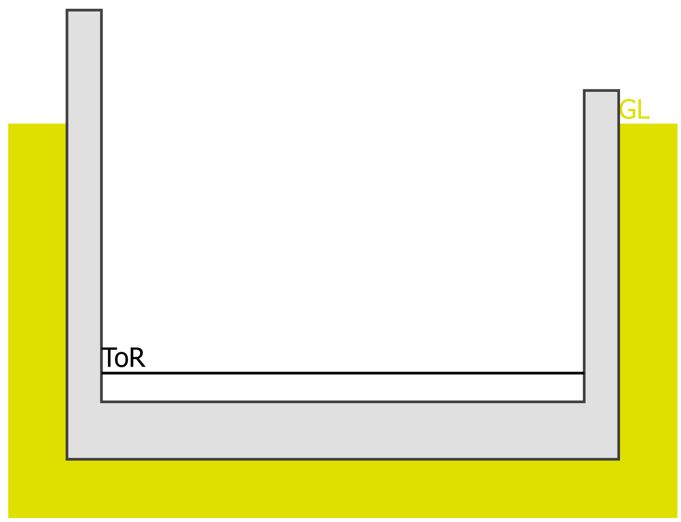
Fresno Grade Separation
Preliminary Design

Section at 10947+50.000
Original Ground Level 287.800
Groundwater Level 235.000
Top of Rail 266.187
Top of Base 263.687
Founding Level 258.687

UN-BRACED U-TROUGH
Trough Depth = 24.113 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 24.113 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 818.831 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	2.00	0.50	0.00
Smaller Bar	2.00	0.50	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.184	2.625
Area (sq-in)	6.676	0.000

Neutral Axis Depth = 9.817 in
Section is Tension controlled.
Reinforcement Strain 6.587e-03
Compression-block depth = 7.854 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 818.836 kip-ft/ft
Shear Checks
Required capacity = 600.858 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.625 in
Shear Reinforcement Area = 3.682 sq in/ft run
Shear Capacity Provided = 850.762 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 335.655 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.875	2.500
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 541.424 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip

SECTION AT 20.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 620.430 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.75	0.00	0.00
Smaller Bar	1.75	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.625	2.500
Area (sq-in)	4.811	0.000

Neutral Axis Depth = 7.074 in
Section is Tension controlled.
Reinforcement Strain 1.041e-02
Compression-block depth = 5.659 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 623.346 kip-ft/ft
Shear Checks
Required capacity = 541.424 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 563.564 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

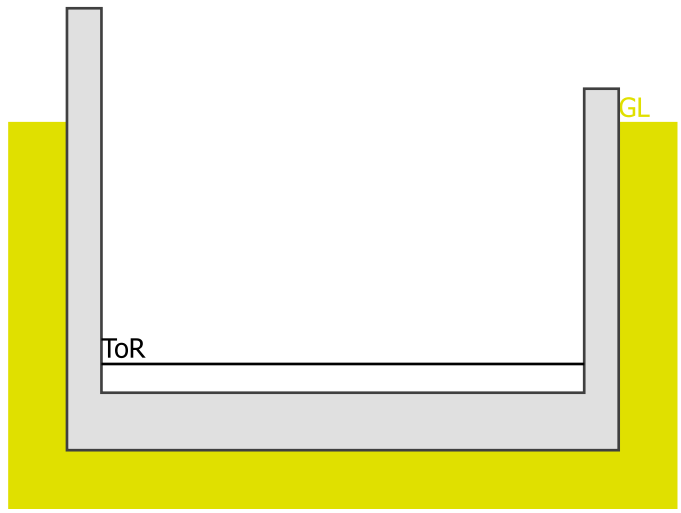
Fresno Grade Separation
Preliminary Design

Section at 10948+ 0.000
Original Ground Level 287.910
Groundwater Level 235.000
Top of Rail 266.962
Top of Base 264.462
Founding Level 259.462

UN-BRACED U-TROUGH
Trough Depth = 23.448 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 23.448 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 783.088 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	2.00	0.25	0.00
Smaller Bar	2.00	0.25	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.327	2.625
Area (sq-in)	6.381	0.000

Neutral Axis Depth = 9.384 in
Section is Tension controlled.
Reinforcement Strain 7.030e-03
Compression-block depth = 7.507 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 791.795 kip-ft/ft
Shear Checks
Required capacity = 597.271 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.625 in
Shear Reinforcement Area = 3.682 sq in/ft run
Shear Capacity Provided = 855.915 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 335.655 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.875	2.500
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 541.424 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip

SECTION AT 20.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 620.430 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.75	0.00	0.00
Smaller Bar	1.75	0.00	0.00

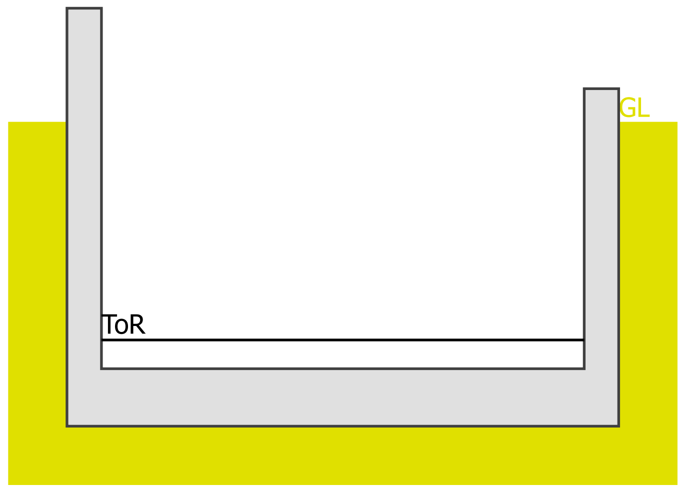
Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.625	2.500
Area (sq-in)	4.811	0.000

Neutral Axis Depth = 7.074 in
Section is Tension controlled.
Reinforcement Strain 1.041e-02
Compression-block depth = 5.659 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 623.346 kip-ft/ft
Shear Checks
Required capacity = 541.424 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 563.564 kip

Section at 10948+50.000
Original Ground Level 286.600
Groundwater Level 235.000
Top of Rail 267.737
Top of Base 265.237
Founding Level 260.237

UN-BRACED U-TROUGH
Trough Depth = 21.363 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 21.363 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 680.308 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.88	0.00	0.00
Smaller Bar	1.88	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.438	2.625
Area (sq-in)	5.522	0.000

Neutral Axis Depth = 8.121 in
Section is Tension controlled.
Reinforcement Strain 8.613e-03
Compression-block depth = 6.497 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 700.512 kip-ft/ft
Shear Checks
Required capacity = 586.532 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.625 in
Shear Reinforcement Area = 3.682 sq in/ft run
Shear Capacity Provided = 857.600 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 335.655 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.875	2.500
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 541.424 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip

SECTION AT 20.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 620.430 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.75	0.00	0.00
Smaller Bar	1.75	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.625	2.500
Area (sq-in)	4.811	0.000

Neutral Axis Depth = 7.074 in
Section is Tension controlled.
Reinforcement Strain 1.041e-02
Compression-block depth = 5.659 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 623.346 kip-ft/ft
Shear Checks
Required capacity = 541.424 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 563.564 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

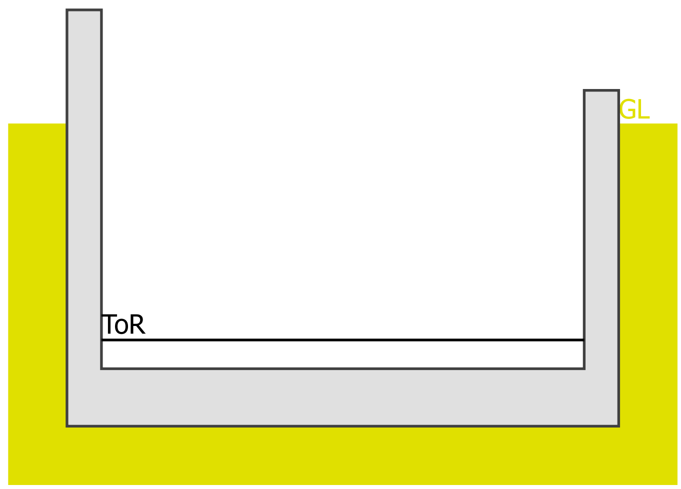
Fresno Grade Separation
Preliminary Design

Section at 10949+ 0.000
Original Ground Level 287.230
Groundwater Level 235.000
Top of Rail 268.512
Top of Base 266.012
Founding Level 261.012

UN-BRACED U-TROUGH
Trough Depth = 21.218 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 21.218 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 673.663 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.88	0.00	0.00
Smaller Bar	1.88	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.438	2.625
Area (sq-in)	5.522	0.000

Neutral Axis Depth = 8.121 in
Section is Tension controlled.
Reinforcement Strain 8.613e-03
Compression-block depth = 6.497 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 700.512 kip-ft/ft
Shear Checks
Required capacity = 585.813 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.625 in
Shear Reinforcement Area = 3.682 sq in/ft run
Shear Capacity Provided = 857.600 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 335.655 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.875	2.500
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 541.424 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip

SECTION AT 20.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 620.430 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.75	0.00	0.00
Smaller Bar	1.75	0.00	0.00

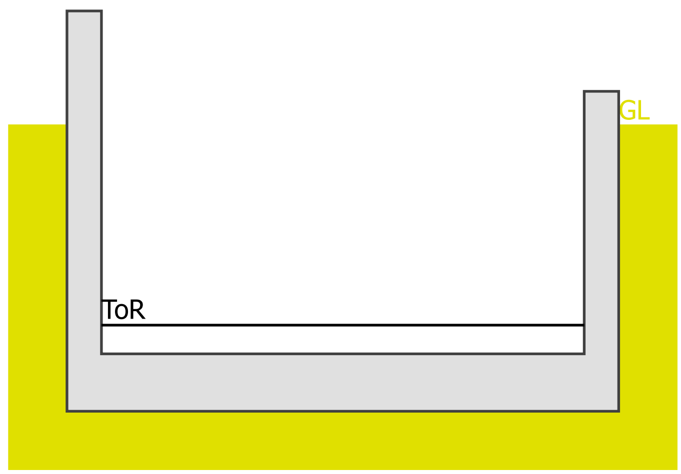
Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.625	2.500
Area (sq-in)	4.811	0.000

Neutral Axis Depth = 7.074 in
Section is Tension controlled.
Reinforcement Strain 1.041e-02
Compression-block depth = 5.659 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 623.346 kip-ft/ft
Shear Checks
Required capacity = 541.424 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 563.564 kip

Section at 10949+50.000
Original Ground Level 286.650
Groundwater Level 235.000
Top of Rail 269.287
Top of Base 266.787
Founding Level 261.787

UN-BRACED U-TROUGH
Trough Depth = 19.863 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 19.863 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 614.717 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.75	0.00	0.00
Smaller Bar	1.75	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.500	2.625
Area (sq-in)	4.811	0.000

Neutral Axis Depth = 7.074 in
Section is Tension controlled.
Reinforcement Strain 1.036e-02
Compression-block depth = 5.659 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 620.640 kip-ft/ft
Shear Checks
Required capacity = 579.277 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.625 in
Shear Reinforcement Area = 3.682 sq in/ft run
Shear Capacity Provided = 859.285 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 335.655 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.875	2.500
Area (sq-in)	2.454	0.000

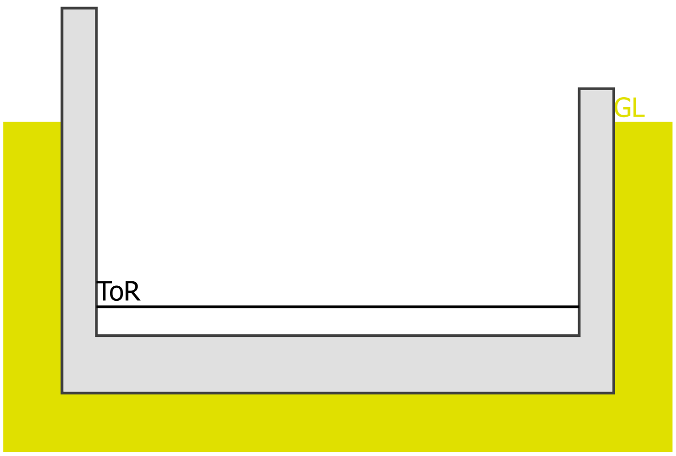
Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 541.424 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip

Section at 10950+ 0.000
Original Ground Level 286.020
Groundwater Level 235.000
Top of Rail 270.062
Top of Base 267.562
Founding Level 262.562

UN-BRACED U-TROUGH
Trough Depth = 18.458 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 18.458 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 559.333 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.75	0.00	0.00
Smaller Bar	1.75	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.500	2.625
Area (sq-in)	4.811	0.000

Neutral Axis Depth = 7.074 in
Section is Tension controlled.
Reinforcement Strain 1.036e-02
Compression-block depth = 5.659 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 620.640 kip-ft/ft
Shear Checks
Required capacity = 572.841 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.625 in
Shear Reinforcement Area = 3.682 sq in/ft run
Shear Capacity Provided = 859.285 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 335.655 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.875	2.500
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.349e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 336.103 kip-ft/ft
Shear Checks
Required capacity = 541.424 kip
Shear Link Spacing L = 2.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.500 in
Shear Reinforcement Area = 2.356 sq in/ft run
Shear Capacity Provided = 567.984 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

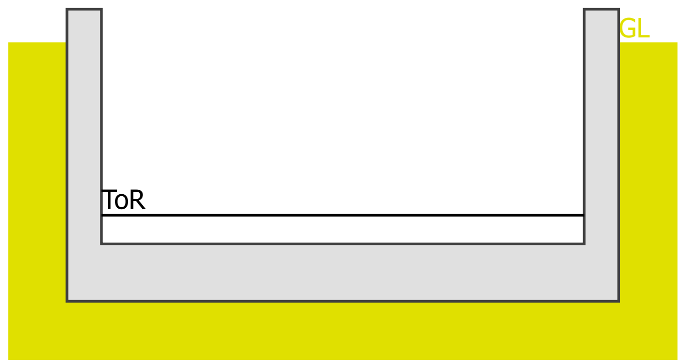
Fresno Grade Separation
Preliminary Design

Section at 10950+50.000
Original Ground Level 285.780
Groundwater Level 235.000
Top of Rail 270.837
Top of Base 268.337
Founding Level 263.337

UN-BRACED U-TROUGH
Trough Depth = 17.443 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 17.443 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 522.846 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.62	0.00	0.00
Smaller Bar	1.62	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.938	2.250
Area (sq-in)	4.148	0.000

Neutral Axis Depth = 6.100 in
Section is Tension controlled.
Reinforcement Strain 1.271e-02
Compression-block depth = 4.880 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 550.586 kip-ft/ft
Shear Checks
Required capacity = 47.922 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 37.912 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 335.655 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.125	2.250
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 20.936 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

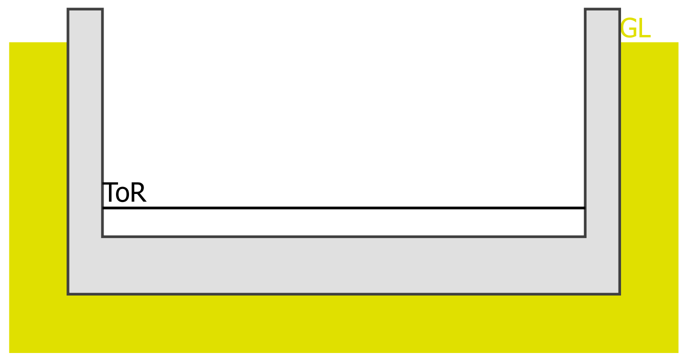
Fresno Grade Separation
Preliminary Design

Section at 10951+ 0.000
Original Ground Level 285.900
Groundwater Level 235.000
Top of Rail 271.597
Top of Base 269.097
Founding Level 264.097

UN-BRACED U-TROUGH
Trough Depth = 16.803 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 16.803 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 501.283 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.62	0.00	0.00
Smaller Bar	1.62	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.938	2.250
Area (sq-in)	4.148	0.000

Neutral Axis Depth = 6.100 in
Section is Tension controlled.
Reinforcement Strain 1.271e-02
Compression-block depth = 4.880 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 550.586 kip-ft/ft
Shear Checks
Required capacity = 45.219 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 37.912 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 335.655 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.125	2.250
Area (sq-in)	2.454	0.000

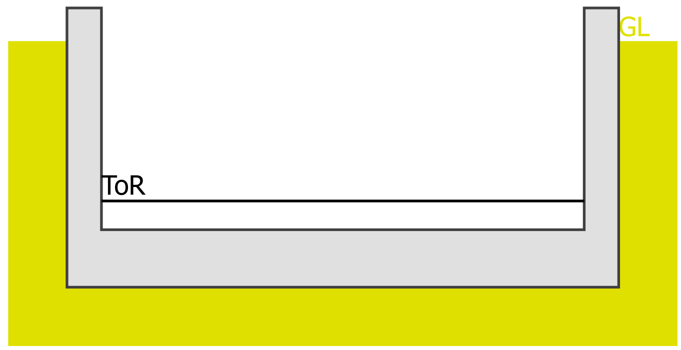
Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 20.936 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

Section at 10951+50.000
Original Ground Level 286.130
Groundwater Level 235.000
Top of Rail 272.338
Top of Base 269.838
Founding Level 264.838

UN-BRACED U-TROUGH
Trough Depth = 16.292 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 16.292 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 484.861 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.62	0.00	0.00
Smaller Bar	1.62	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	31.938	2.250
Area (sq-in)	4.148	0.000

Neutral Axis Depth = 6.100 in
Section is Tension controlled.
Reinforcement Strain 1.271e-02
Compression-block depth = 4.880 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 550.586 kip-ft/ft
Shear Checks
Required capacity = 43.113 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 37.912 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 335.655 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

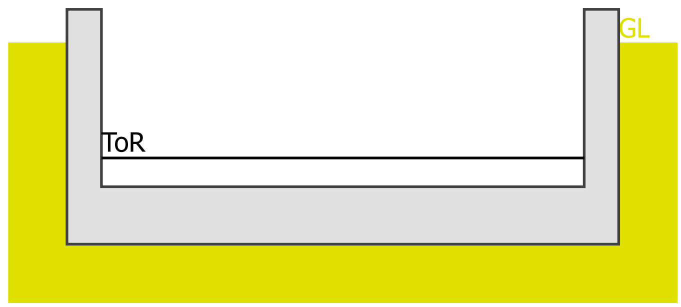
Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.125	2.250
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 20.936 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

Section at 10952+ 0.000
Original Ground Level 282.990
Groundwater Level 235.000
Top of Rail 273.059
Top of Base 270.559
Founding Level 265.559



UN-BRACED U-TROUGH
Trough Depth = 12.431 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf

WALL ROOT SECTION at 12.431 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 382.264 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.38	0.00	0.00
Smaller Bar	1.38	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.062	2.250
Area (sq-in)	2.970	0.000

Neutral Axis Depth = 4.367 in
Section is Tension controlled.
Reinforcement Strain 1.902e-02
Compression-block depth = 3.494 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 405.138 kip-ft/ft
Shear Checks
Required capacity = 28.678 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.061 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 335.655 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.125	2.250
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 20.936 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

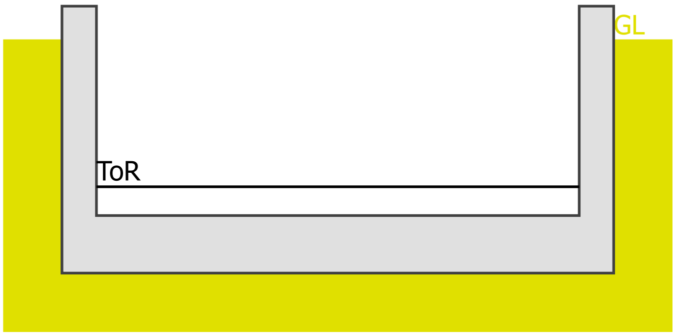
Fresno Grade Separation
Preliminary Design

Section at 10952+50.000
Original Ground Level 286.450
Groundwater Level 235.000
Top of Rail 273.762
Top of Base 271.262
Founding Level 266.262

UN-BRACED U-TROUGH
Trough Depth = 15.188 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 15.188 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 451.708 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.50	0.00	0.00
Smaller Bar	1.50	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.000	2.250
Area (sq-in)	3.534	0.000

Neutral Axis Depth = 5.197 in
Section is Tension controlled.
Reinforcement Strain 1.547e-02
Compression-block depth = 4.158 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 475.873 kip-ft/ft
Shear Checks
Required capacity = 38.717 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 37.987 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 335.655 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.125	2.250
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 20.936 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08

Designed by : AJA

Checked by : YR/SS

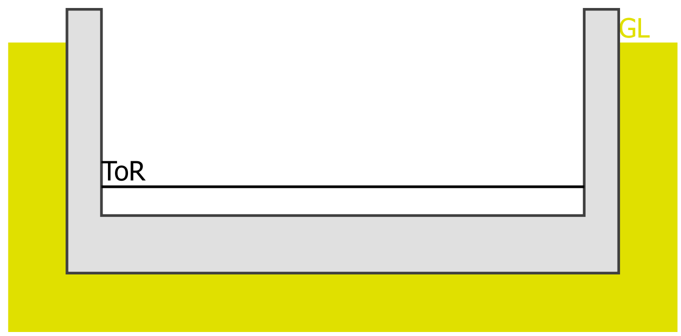
Fresno Grade Separation
Preliminary Design

Section at 10953+ 0.000
Original Ground Level 286.860
Groundwater Level 235.000
Top of Rail 274.446
Top of Base 271.946
Founding Level 266.946

UN-BRACED U-TROUGH
Trough Depth = 14.914 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 14.914 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 443.981 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.50	0.00	0.00
Smaller Bar	1.50	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.000	2.250
Area (sq-in)	3.534	0.000

Neutral Axis Depth = 5.197 in
Section is Tension controlled.
Reinforcement Strain 1.547e-02
Compression-block depth = 4.158 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 475.873 kip-ft/ft
Shear Checks
Required capacity = 37.661 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 37.987 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 335.655 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

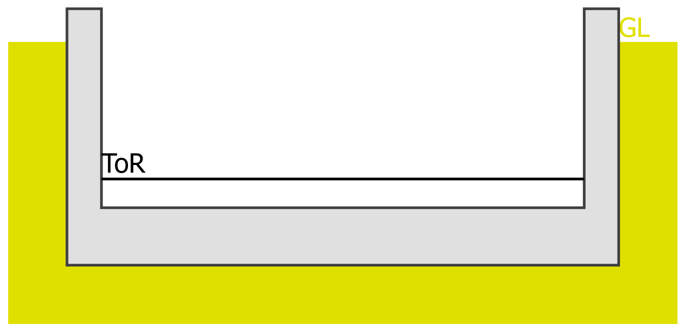
Layer	Tension	Compression
Depth (in)	32.125	2.250
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 20.936 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

Fresno Grade Separation
Preliminary Design

Section at 10953+50.000
Original Ground Level 286.920
Groundwater Level 235.000
Top of Rail 275.110
Top of Base 272.610
Founding Level 267.610

UN-BRACED U-TROUGH
Trough Depth = 14.310 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 14.310 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 427.577 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.50	0.00	0.00
Smaller Bar	1.50	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.000	2.250
Area (sq-in)	3.534	0.000

Neutral Axis Depth = 5.197 in
Section is Tension controlled.
Reinforcement Strain 1.547e-02
Compression-block depth = 4.158 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 475.873 kip-ft/ft
Shear Checks
Required capacity = 35.375 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 37.987 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 335.655 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.125	2.250
Area (sq-in)	2.454	0.000

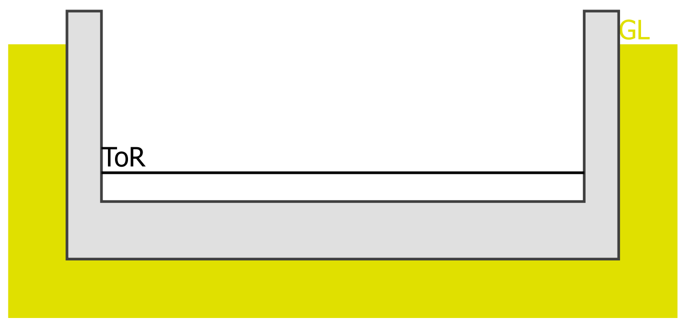
Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 20.936 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10954+ 0.000
Original Ground Level 286.800
Groundwater Level 235.000
Top of Rail 275.756
Top of Base 273.256
Founding Level 268.256

UN-BRACED U-TROUGH
Trough Depth = 13.544 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 13.544 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 408.101 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.50	0.00	0.00
Smaller Bar	1.50	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.000	2.250
Area (sq-in)	3.534	0.000

Neutral Axis Depth = 5.197 in
Section is Tension controlled.
Reinforcement Strain 1.547e-02
Compression-block depth = 4.158 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 475.873 kip-ft/ft
Shear Checks
Required capacity = 32.573 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 37.987 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 335.655 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.125	2.250
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 20.936 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

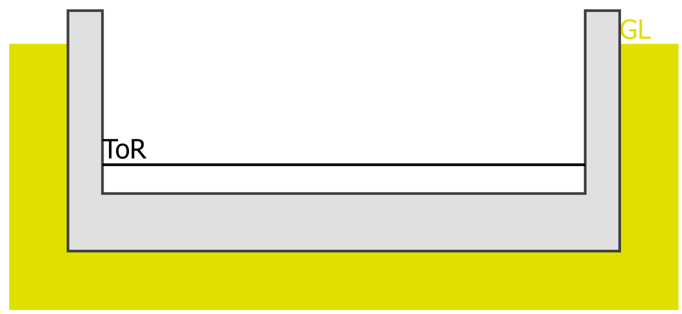
Fresno Grade Separation
Preliminary Design

Section at 10954+50.000
Original Ground Level 286.770
Groundwater Level 235.000
Top of Rail 276.382
Top of Base 273.882
Founding Level 268.882

UN-BRACED U-TROUGH
Trough Depth = 12.888 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 12.888 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 392.525 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.38	0.00	0.00
Smaller Bar	1.38	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.062	2.250
Area (sq-in)	2.970	0.000

Neutral Axis Depth = 4.367 in
Section is Tension controlled.
Reinforcement Strain 1.902e-02
Compression-block depth = 3.494 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 405.138 kip-ft/ft
Shear Checks
Required capacity = 30.252 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.061 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 335.655 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.125	2.250
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 20.936 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

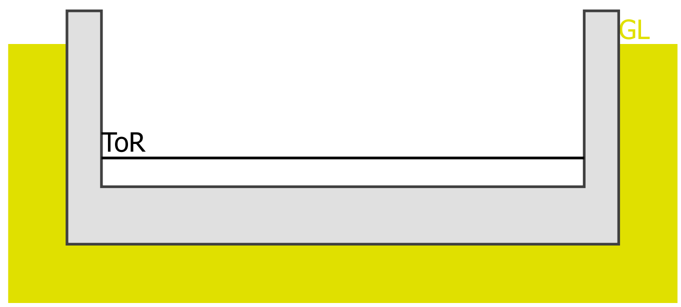
California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10955+ 0.000
Original Ground Level 286.790
Groundwater Level 235.000
Top of Rail 276.989
Top of Base 274.489
Founding Level 269.489

UN-BRACED U-TROUGH
Trough Depth = 12.301 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 12.301 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 379.446 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.38	0.00	0.00
Smaller Bar	1.38	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.062	2.250
Area (sq-in)	2.970	0.000

Neutral Axis Depth = 4.367 in
Section is Tension controlled.
Reinforcement Strain 1.902e-02
Compression-block depth = 3.494 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 405.138 kip-ft/ft
Shear Checks
Required capacity = 28.239 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.061 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 335.655 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.125	2.250
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 20.936 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

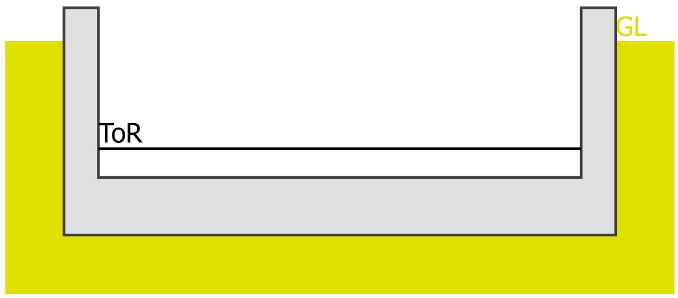
Fresno Grade Separation
Preliminary Design

Section at 10955+50.000
Original Ground Level 286.820
Groundwater Level 235.000
Top of Rail 277.577
Top of Base 275.077
Founding Level 270.077

UN-BRACED U-TROUGH
Trough Depth = 11.743 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 11.743 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 367.750 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.38	0.00	0.00
Smaller Bar	1.38	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.062	2.250
Area (sq-in)	2.970	0.000

Neutral Axis Depth = 4.367 in
Section is Tension controlled.
Reinforcement Strain 1.902e-02
Compression-block depth = 3.494 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 405.138 kip-ft/ft
Shear Checks
Required capacity = 26.383 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.061 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 335.655 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

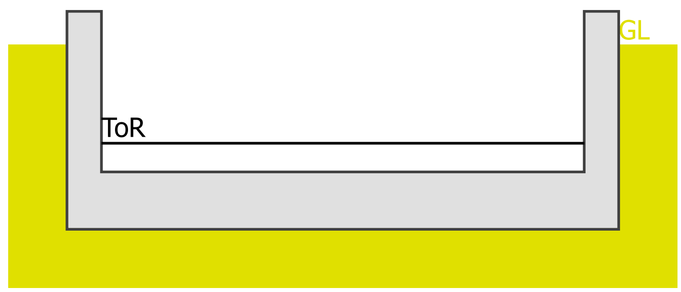
Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.125	2.250
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 20.936 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

Section at 10956+ 0.000
Original Ground Level 286.640
Groundwater Level 235.000
Top of Rail 278.146
Top of Base 275.646
Founding Level 270.646

UN-BRACED U-TROUGH
Trough Depth = 10.994 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 10.994 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 353.150 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.38	0.00	0.00
Smaller Bar	1.38	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.062	2.250
Area (sq-in)	2.970	0.000

Neutral Axis Depth = 4.367 in
Section is Tension controlled.
Reinforcement Strain 1.902e-02
Compression-block depth = 3.494 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 405.138 kip-ft/ft
Shear Checks
Required capacity = 23.977 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.061 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 335.655 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

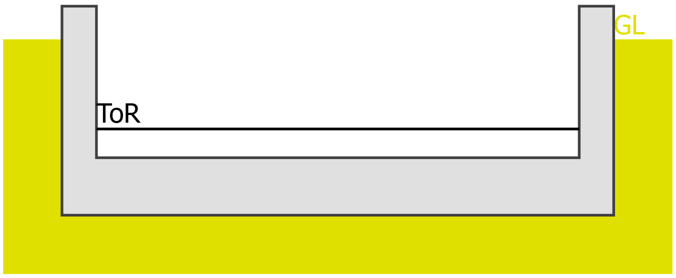
Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.125	2.250
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 20.936 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

Section at 10956+50.000
Original Ground Level 286.380
Groundwater Level 235.000
Top of Rail 278.696
Top of Base 276.196
Founding Level 271.196

UN-BRACED U-TROUGH
Trough Depth = 10.184 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 10.184 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 338.739 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.25 0.00 0.00
Smaller Bar 1.25 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.125 2.250
Area (sq-in) 2.454 0.000
Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 21.487 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip
SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 335.655 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.25 0.00 0.00
Smaller Bar 1.25 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.125 2.250
Area (sq-in) 2.454 0.000
Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 20.936 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

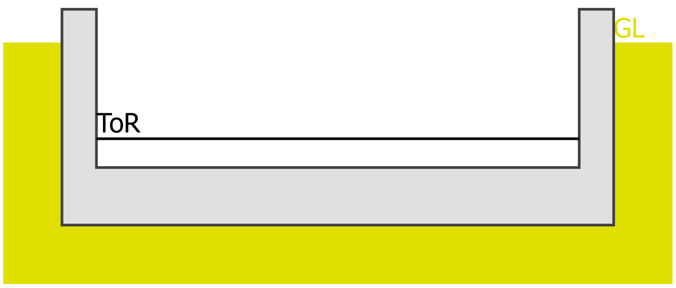
Fresno Grade Separation
Preliminary Design

Section at 10957+ 0.000
Original Ground Level 287.470
Groundwater Level 235.000
Top of Rail 279.226
Top of Base 276.726
Founding Level 271.726

UN-BRACED U-TROUGH
Trough Depth = 10.744 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 10.744 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 348.543 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.38	0.00	0.00
Smaller Bar	1.38	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.062	2.250
Area (sq-in)	2.970	0.000

Neutral Axis Depth = 4.367 in
Section is Tension controlled.
Reinforcement Strain 1.902e-02
Compression-block depth = 3.494 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 405.138 kip-ft/ft
Shear Checks
Required capacity = 23.195 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.061 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 335.655 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.125	2.250
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 20.936 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

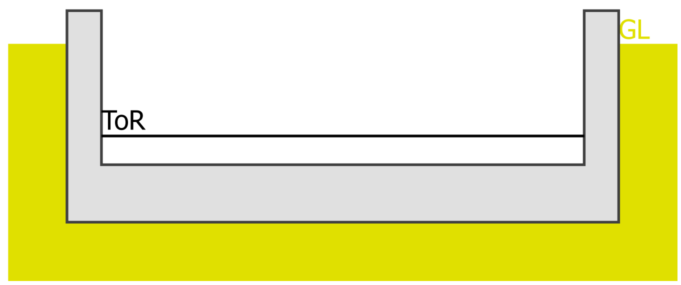
Fresno Grade Separation
Preliminary Design

Section at 10957+50.000
Original Ground Level 287.640
Groundwater Level 235.000
Top of Rail 279.738
Top of Base 277.238
Founding Level 272.238

UN-BRACED U-TROUGH
Trough Depth = 10.402 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 10.402 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 342.477 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.38	0.00	0.00
Smaller Bar	1.38	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.062	2.250
Area (sq-in)	2.970	0.000

Neutral Axis Depth = 4.367 in
Section is Tension controlled.
Reinforcement Strain 1.902e-02
Compression-block depth = 3.494 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 405.138 kip-ft/ft
Shear Checks
Required capacity = 22.145 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.061 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 335.655 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

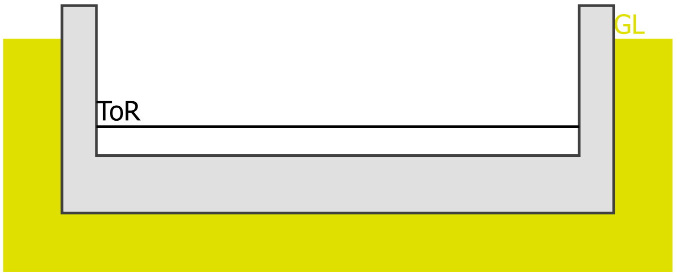
Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.125	2.250
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 20.936 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

Section at 10958+ 0.000
Original Ground Level 287.750
Groundwater Level 235.000
Top of Rail 280.230
Top of Base 277.730
Founding Level 272.730

UN-BRACED U-TROUGH
Trough Depth = 10.020 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 10.020 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 335.979 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.125	2.250
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 20.995 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

SECTION AT 10.000 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 335.655 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.125	2.250
Area (sq-in)	2.454	0.000

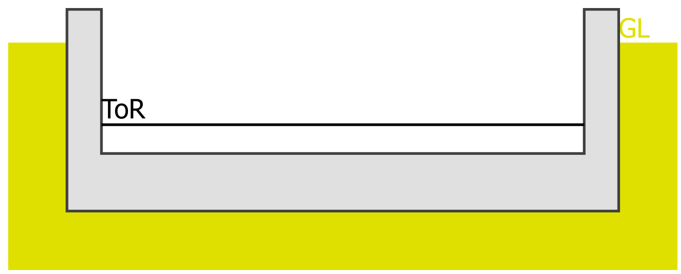
Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 20.936 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

Section at 10958+50.000
Original Ground Level 287.720
Groundwater Level 235.000
Top of Rail 280.704
Top of Base 278.204
Founding Level 273.204

UN-BRACED U-TROUGH
Trough Depth = 9.516 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 9.516 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 327.894 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.125	2.250
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 19.519 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10959+ 0.000
Original Ground Level 286.900
Groundwater Level 235.000
Top of Rail 281.158
Top of Base 278.658
Founding Level 273.658

UN-BRACED U-TROUGH
Trough Depth = 8.242 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf

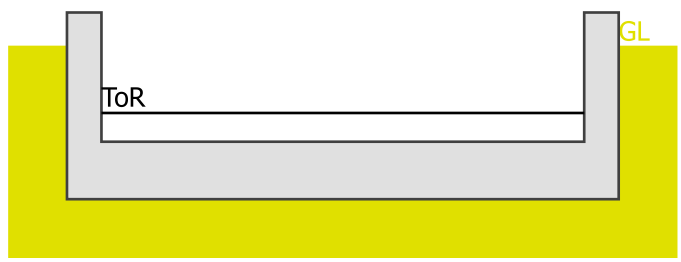
WALL ROOT SECTION at 8.242 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 309.724 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.125	2.250
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 15.984 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip



Fresno Grade Separation
Preliminary Design

Section at 10959+50.000
Original Ground Level 286.340
Groundwater Level 235.000
Top of Rail 281.593
Top of Base 279.093
Founding Level 274.093

UN-BRACED U-TROUGH
Trough Depth = 7.247 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf

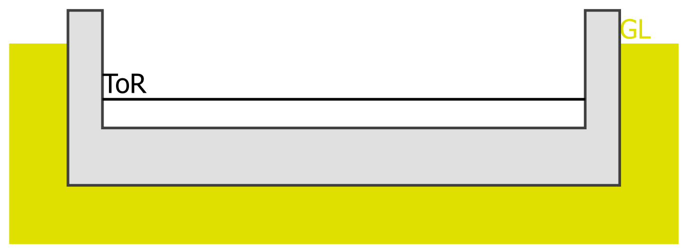
WALL ROOT SECTION at 7.247 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 297.733 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.125	2.250
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 13.421 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip



California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10960+ 0.000
Original Ground Level 286.810
Groundwater Level 235.000
Top of Rail 282.009
Top of Base 279.509
Founding Level 274.509

UN-BRACED U-TROUGH
Trough Depth = 7.301 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf

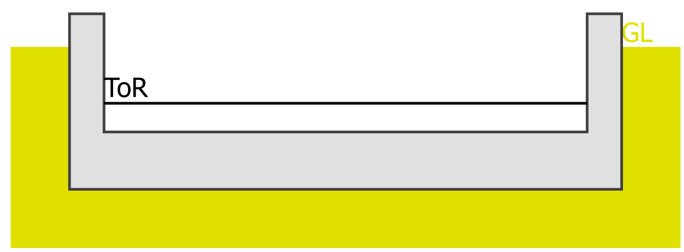
WALL ROOT SECTION at 7.301 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 298.336 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.125	2.250
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 13.555 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip



Fresno Grade Separation
Preliminary Design

Section at 10960+50.000
Original Ground Level 286.750
Groundwater Level 235.000
Top of Rail 282.406
Top of Base 279.906
Founding Level 274.906

UN-BRACED U-TROUGH
Trough Depth = 6.844 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf

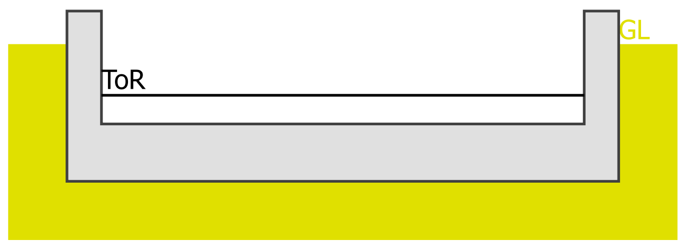
WALL ROOT SECTION at 6.844 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 293.405 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.125	2.250
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 12.433 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip



Section at 10961+ 0.000
Original Ground Level 286.620
Groundwater Level 235.000
Top of Rail 282.784
Top of Base 280.284
Founding Level 275.284

UN-BRACED U-TROUGH
Trough Depth = 6.336 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf

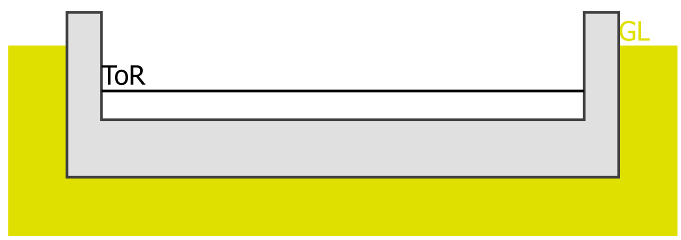
WALL ROOT SECTION at 6.336 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 288.369 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.125	2.250
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 11.228 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip



California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10961+50.000
Original Ground Level 286.220
Groundwater Level 235.000
Top of Rail 283.142
Top of Base 280.642
Founding Level 275.642

UN-BRACED U-TROUGH
Trough Depth = 5.578 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf

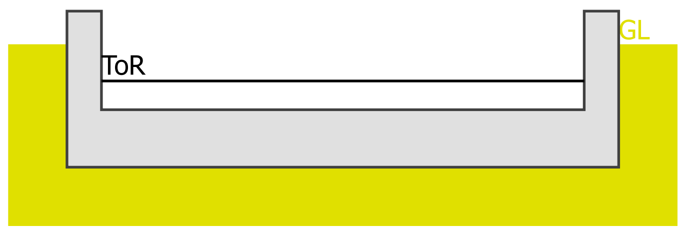
WALL ROOT SECTION at 5.578 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 281.692 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.125	2.250
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 9.512 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip



California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10962+ 0.000
Original Ground Level 286.000
Groundwater Level 235.000
Top of Rail 283.482
Top of Base 280.982
Founding Level 275.982

UN-BRACED U-TROUGH
Trough Depth = 5.018 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf

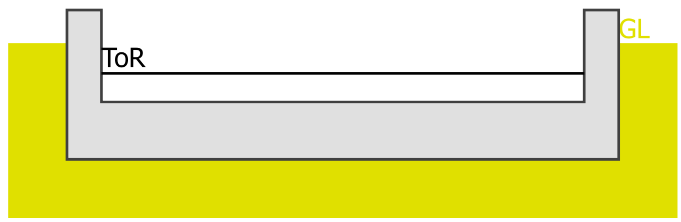
WALL ROOT SECTION at 5.018 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 277.399 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

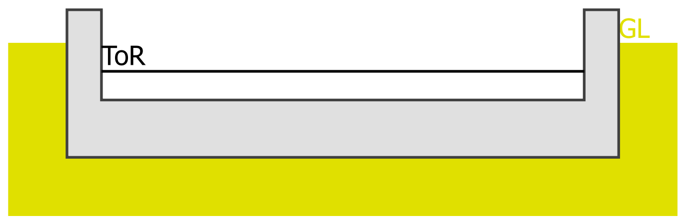
Layer	Tension	Compression
Depth (in)	32.188	2.250
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 8.311 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.209 kip



Section at 10962+50.000
Original Ground Level 286.170
Groundwater Level 235.000
Top of Rail 283.802
Top of Base 281.302
Founding Level 276.302

UN-BRACED U-TROUGH
Trough Depth = 4.868 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 4.868 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 276.334 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.188	2.250
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 7.998 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.209 kip

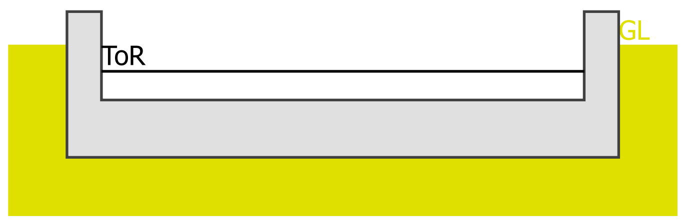
Fresno Grade Separation
Preliminary Design

Section at 10963+ 0.000
Original Ground Level 286.310
Groundwater Level 235.000
Top of Rail 284.103
Top of Base 281.603
Founding Level 276.603

UN-BRACED U-TROUGH
Trough Depth = 4.707 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 4.707 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 275.234 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.188	2.250
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 7.666 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.209 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10963+50.000
Original Ground Level 286.020
Groundwater Level 235.000
Top of Rail 284.385
Top of Base 281.885
Founding Level 276.885

UN-BRACED U-TROUGH
Trough Depth = 4.135 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf

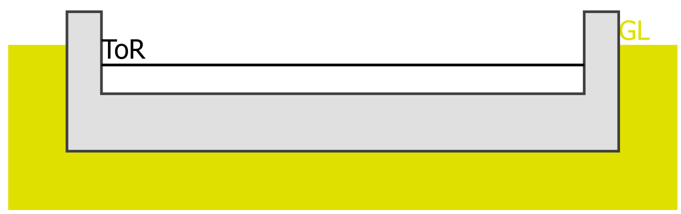
WALL ROOT SECTION at 4.135 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 271.668 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.188	2.250
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 6.527 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.209 kip



Fresno Grade Separation
Preliminary Design

Section at 10964+ 0.000
Original Ground Level 286.450
Groundwater Level 235.000
Top of Rail 284.648
Top of Base 282.148
Founding Level 277.148

UN-BRACED U-TROUGH
Trough Depth = 4.302 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf

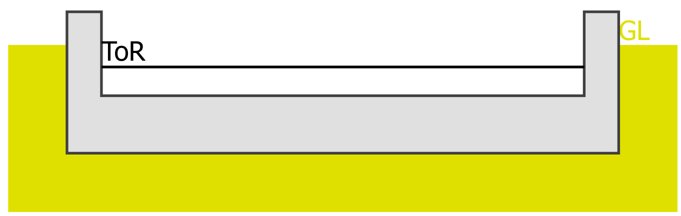
WALL ROOT SECTION at 4.302 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 272.654 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.188	2.250
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 6.853 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.209 kip



California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10964+50.000
Original Ground Level 286.740
Groundwater Level 235.000
Top of Rail 284.892
Top of Base 282.392
Founding Level 277.392

UN-BRACED U-TROUGH
Trough Depth = 4.348 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf

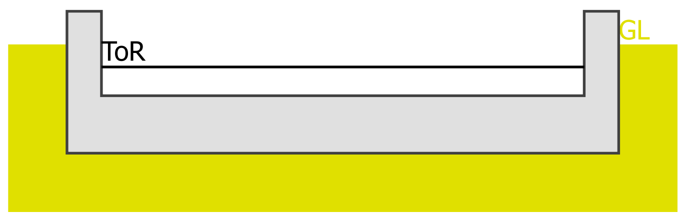
WALL ROOT SECTION at 4.348 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 272.935 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.188	2.250
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 6.944 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.209 kip



California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

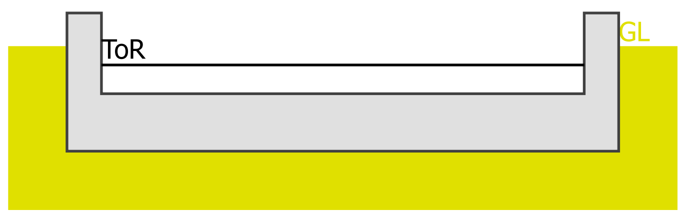
Fresno Grade Separation Preliminary Design

Section at 10965+ 0.000
Original Ground Level 286.640
Groundwater Level 235.000
Top of Rail 285.117
Top of Base 282.617
Founding Level 277.617

UN-BRACED U-TROUGH
Trough Depth = 4.023 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 4.023 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 271.032 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.188	2.250
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 6.311 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.209 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10965+50.000
Original Ground Level 287.020
Groundwater Level 235.000
Top of Rail 285.323
Top of Base 282.823
Founding Level 277.823

UN-BRACED U-TROUGH
Trough Depth = 4.197 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf

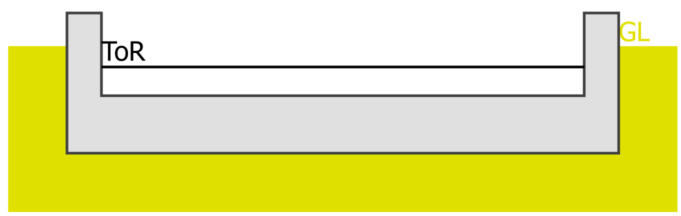
WALL ROOT SECTION at 4.197 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 272.033 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.188	2.250
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 6.649 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.209 kip



Fresno Grade Separation
Preliminary Design

Section at 10966+ 0.000
Original Ground Level 290.020
Groundwater Level 235.000
Top of Rail 285.509
Top of Base 283.009
Founding Level 278.009

UN-BRACED U-TROUGH
Trough Depth = 7.011 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf

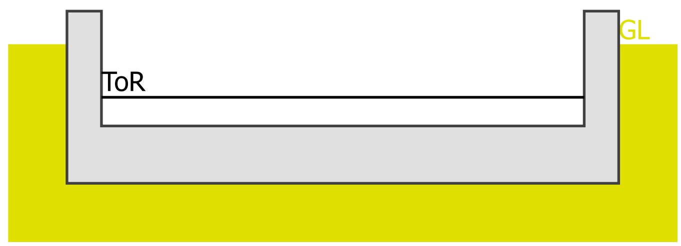
WALL ROOT SECTION at 7.011 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 295.157 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.25	0.00	0.00
Smaller Bar	1.25	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.125	2.250
Area (sq-in)	2.454	0.000

Neutral Axis Depth = 3.609 in
Section is Tension controlled.
Reinforcement Strain 2.370e-02
Compression-block depth = 2.887 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 338.864 kip-ft/ft
Shear Checks
Required capacity = 12.837 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.135 kip



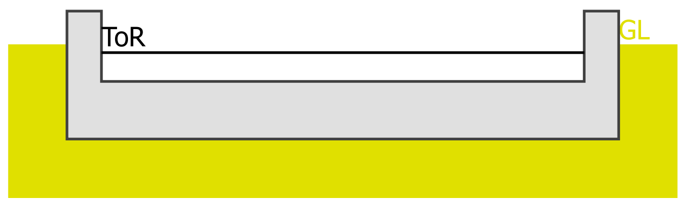
Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation Preliminary Design

Section at 10966+50.000
Original Ground Level 286.280
Groundwater Level 235.000
Top of Rail 285.677
Top of Base 283.177
Founding Level 278.177

UN-BRACED U-TROUGH
Trough Depth = 3.103 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors
DL EH WA LLS CL
1.25 1.35 1.60 1.75 1.00
Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf

WALL ROOT SECTION at 3.103 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 266.534 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in
Layer T1 T2 C1
Larger Bar 1.12 0.00 0.00
Smaller Bar 1.12 0.00 0.00
Cracking Moment = 178.707 kip-ft
Layer Tension Compression
Depth (in) 32.188 2.250
Area (sq-in) 1.988 0.000
Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 4.618 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.209 kip

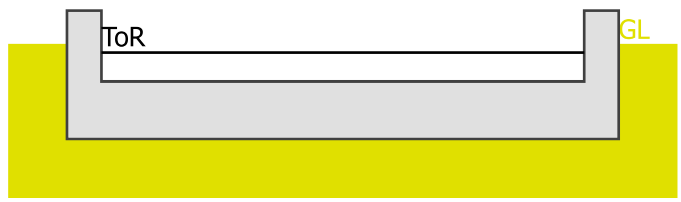


Section at 10967+ 0.000
Original Ground Level 286.470
Groundwater Level 235.000
Top of Rail 285.825
Top of Base 283.325
Founding Level 278.325

UN-BRACED U-TROUGH
Trough Depth = 3.145 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 3.145 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 266.710 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.188	2.250
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 4.691 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.209 kip

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10967+50.000
Original Ground Level 286.260
Groundwater Level 235.000
Top of Rail 285.955
Top of Base 283.455
Founding Level 278.455

UN-BRACED U-TROUGH
Trough Depth = 2.805 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf

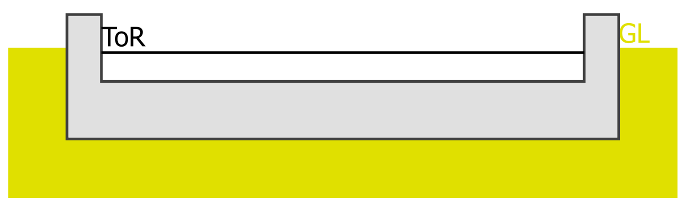
WALL ROOT SECTION at 2.805 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 265.349 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.188	2.250
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 4.100 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.209 kip



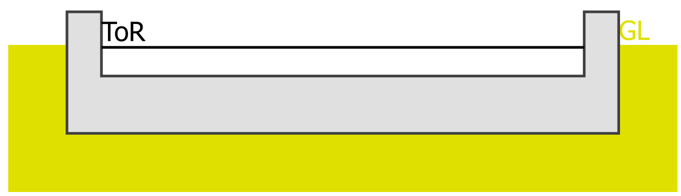
Fresno Grade Separation
Preliminary Design

Section at 10968+ 0.000
Original Ground Level 286.180
Groundwater Level 235.000
Top of Rail 286.080
Top of Base 283.580
Founding Level 278.580

UN-BRACED U-TROUGH
Trough Depth = 2.600 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 2.600 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 264.609 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.188	2.250
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 3.754 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.209 kip

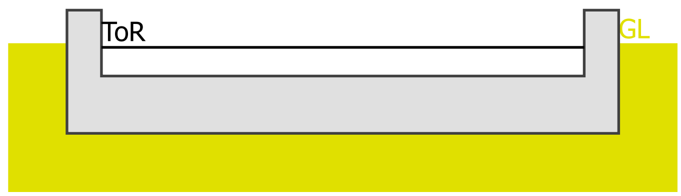
Fresno Grade Separation
Preliminary Design

Section at 10968+50.000
Original Ground Level 286.450
Groundwater Level 235.000
Top of Rail 286.205
Top of Base 283.705
Founding Level 278.705

UN-BRACED U-TROUGH
Trough Depth = 2.745 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 2.745 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 265.126 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.188	2.250
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 3.998 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.209 kip

California High Speed Train
Fresno to Bakersfield
Package 1A

Date : 2011-12-08
Designed by : AJA
Checked by : YR/SS

Fresno Grade Separation
Preliminary Design

Section at 10969+ 0.000
Original Ground Level 286.720
Groundwater Level 235.000
Top of Rail 286.330
Top of Base 283.830
Founding Level 278.830

UN-BRACED U-TROUGH
Trough Depth = 2.890 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf

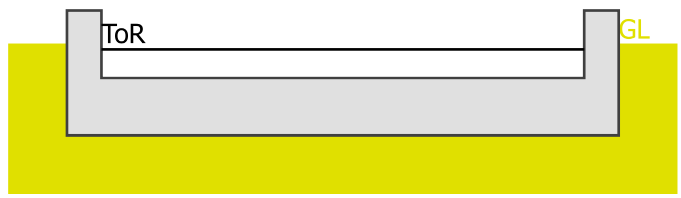
WALL ROOT SECTION at 2.890 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 265.673 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.188	2.250
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 4.246 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.209 kip



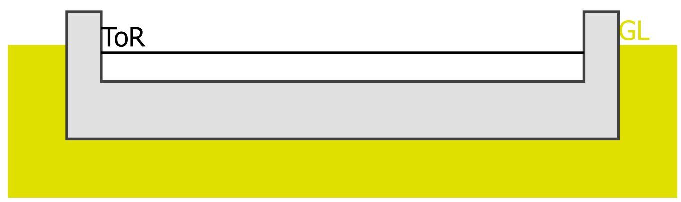
Fresno Grade Separation
Preliminary Design

Section at 10969+50.000
Original Ground Level 287.020
Groundwater Level 235.000
Top of Rail 286.455
Top of Base 283.955
Founding Level 278.955

UN-BRACED U-TROUGH
Trough Depth = 3.065 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf



WALL ROOT SECTION at 3.065 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 266.375 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.188	2.250
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 4.550 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.209 kip

Section at 10970+ 0.000
Original Ground Level 286.960
Groundwater Level 235.000
Top of Rail 286.580
Top of Base 284.080
Founding Level 279.080

UN-BRACED U-TROUGH
Trough Depth = 2.880 ft
Base Thickness = 5.000 ft
Min Wall Thickness = 3.000 ft
Trough internal width = 42.000 ft
Design Load factors

DL	EH	WA	LLS	CL
1.25	1.35	1.60	1.75	1.00

Live Load Surcharge = 600.000 psf
FoS Against Flotation = Inf

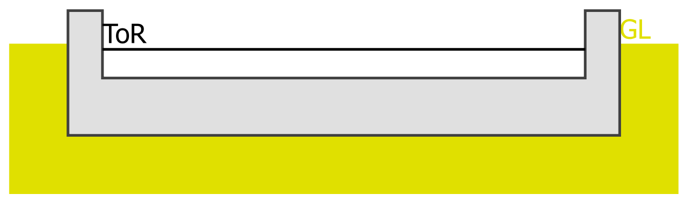
WALL ROOT SECTION at 2.880 BELOW GROUND
RC SECTION DESIGN
Bending Checks
Required capacity = 265.634 kip-ft
Section thickness = 36.000 in
Bar Spacing = 6.00 in

Layer	T1	T2	C1
Larger Bar	1.12	0.00	0.00
Smaller Bar	1.12	0.00	0.00

Cracking Moment = 178.707 kip-ft

Layer	Tension	Compression
Depth (in)	32.188	2.250
Area (sq-in)	1.988	0.000

Neutral Axis Depth = 2.924 in
Section is Tension controlled.
Reinforcement Strain 3.003e-02
Compression-block depth = 2.339 in
Resistance factor (Phi) = 0.90
Moment Capacity (Phi.Mn) = 277.493 kip-ft/ft
Shear Checks
Required capacity = 4.229 kip
Shear Link Spacing L = 6.000 in
Shear Link Spacing T = 6.000 in
Shear Link Diameter = 0.250 in
Shear Reinforcement Area = 0.000 sq in/ft run
Shear Capacity Provided = 38.209 kip



Brace Calculations

Check of Permanent Brace Section

Section Dimensions

Width $b := 1.5 \cdot ft$

Depth $h := 3 \cdot ft$

Weight of Section

Concrete density $\gamma_c := 160 \cdot \frac{lb}{ft^3}$

Gross Area $A := b \cdot h$ $\quad A = 4.5 \cdot ft^2$

Span of Prop $L := 43 \cdot ft$

Weight per foot $W := \gamma_c \cdot A \cdot g$ $\quad W = 720 \cdot \frac{lbf}{ft}$

Moment due to self weight $M_{sw} := \frac{W \cdot L^2}{8}$ $\quad M_{sw} = 166.41 \cdot kip \cdot ft$

Live Load

Allow for a point load at mid span to represent the possibility that a vehicle might get through that barriers and fall in to the trench, resting on top of the props!

$P_{LL} := 36 \cdot kip$ $\quad M_{LL} := P_{LL} \cdot \frac{L}{4}$ $\quad M_{LL} = 387 \cdot kip \cdot ft$

Deflection of Beam

$E_c := 5000 \cdot \frac{kip}{in^2}$

Allow for Long term creep effects $E_e := \frac{E_c}{2}$ $\quad E_e = 2.5 \times 10^3 \cdot \frac{kip}{in^2}$

$I := b \cdot \frac{h^3}{12}$

Allow for cracked concrete stiffness $I_c := I \cdot 0.4$ $\quad I = 6.998 \times 10^4 \cdot in^4$

$$\delta := \frac{5 \cdot W \cdot L^4}{384 \cdot E_e \cdot I_c} + \frac{P_{LL} \cdot L^3}{192 \cdot E_e \cdot I_c}$$

$\delta = 1.159 \cdot in$

Earth Pressures

From trench Analysis force in brace for a 20ft spacing is $P := 1235 \cdot kip$

Additional P - Delta Moment $M_a := P \cdot \delta \cdot 1.5$

Extra 1.5 factor because this should be iterative. $\quad M_a = 178.994 \cdot kip \cdot ft$

Note: Consider P - Delta to be an Earth Pressure Force

Load Factors

Importance classifications

$\eta_I := 1.05$ Critical structure for HST

$\eta_R := 1.05$ Non - redundant

$\eta_D := 1.05$ Non - ductile

$\eta := \eta_I \cdot \eta_R \cdot \eta_D$ $\quad \eta = 1.158$

Check of Permanent Brace Section

Dead Load	$\gamma_{DC} := 1.25$
Super Dead Load	$\gamma_{DW} := 1.5$
Live Load	$\gamma_{LL} := 1.75$
Earth Pressure	$\gamma_{EH} := 1.35$
Locked IN Force	$\gamma_{EL} := 1.0$

$Q := \eta \cdot \left(M_{sw} \cdot \gamma_{DC} + M_{LL} \cdot \gamma_{LL} + M_a \cdot \gamma_{EH}\right)$ $\qquad Q = 1.305 \times 10^3 \cdot kip \cdot ft$

Capacity from ADSEC Calculation is $\qquad M_R := 1569 \cdot kip \cdot ft$

Section Utilization $\qquad U := \dfrac{Q}{M_R}$

$ans := if(U < 0.95, "OK", "Not OK") \qquad ans = "OK"$

Elastic shortening of props under load (Long term E)

$\Delta := \dfrac{P \cdot L}{E_e \cdot A} \qquad \Delta = 0.393 \cdot in$

Ensure that the wall design can accommodate this movement over time

CHST Fresno - Bakersfield

Trough Brace Design Check

History

Date	Time	Name	Note
27-Nov-2011	21:50	Andrew.Armstrong	New
01-Dec-2011	16:44	andrew.armstrong	
01-Dec-2011	18:01	andrew.armstrong	Save as J:\S-F\131000\131577\4 Internal Project Data\4-04 Calculations\Struct\30% Calculations\Fresno Trench\retwall section analysis.ads
07-Dec-2011	22:54	Andrew.Armstrong	

Specification

General Specification

Code of Practice	ACI 318-08
Country	United States
Bending Axes	Biaxial
Strength reduction ϕ calculated using:	ACI 318 Fig. R9.3.2

Section 1 Details

Definition

Name	Brace
Type	Concrete
Material	6000 psi
Origin	Centre
Dimensions	
Depth	36.00in
Width	18.00in
Section Area	648.0in^2
Reinforcement Area	30.95in^2
Reinforcement	4.776%

Section Nodes

Node	Y [in]	Z [in]
1	9.000	18.00
2	9.000	-18.00
3	-9.000	-18.00
4	-9.000	18.00

Cover and Links

Links Discrete

Bars

Bar	Y [in]	Z [in]	Diameter [in]	Material	Type	Pre-stress Force [kip]	Pre-stress Strain	Appl. loads include/exclude pre-stress
1	-7.000	-14.00	1.272	Grade 60	Steel			
2	-2.333	-14.00	1.272	Grade 60	Steel			
3	2.333	-14.00	1.272	Grade 60	Steel			
4	7.000	-14.00	1.272	Grade 60	Steel			
5	-7.000	15.00	1.000	Grade 60	Steel			
6	-4.200	15.00	1.000	Grade 60	Steel			
7	-1.400	15.00	1.000	Grade 60	Steel			
8	1.400	15.00	1.000	Grade 60	Steel			
9	4.200	15.00	1.000	Grade 60	Steel			
10	7.000	15.00	1.000	Grade 60	Steel			
11	-7.000	-12.00	1.272	Grade 60	Steel			
12	-2.333	-12.00	1.272	Grade 60	Steel			
13	2.333	-12.00	1.272	Grade 60	Steel			
14	7.000	-12.00	1.272	Grade 60	Steel			
15	-7.000	12.00	1.000	Grade 60	Steel			
16	-4.200	12.00	1.000	Grade 60	Steel			
17	-1.400	12.00	1.000	Grade 60	Steel			
18	1.400	12.00	1.000	Grade 60	Steel			
19	4.200	12.00	1.000	Grade 60	Steel			
20	7.000	12.00	1.000	Grade 60	Steel			
21	-7.000	10.00	1.000	Grade 60	Steel			
22	7.000	10.00	1.000	Grade 60	Steel			
23	-7.000	8.000	1.000	Grade 60	Steel			
24	7.000	8.000	1.000	Grade 60	Steel			
25	-7.000	6.000	1.000	Grade 60	Steel			
26	7.000	6.000	1.000	Grade 60	Steel			
27	-7.000	4.000	1.000	Grade 60	Steel			
28	7.000	4.000	1.000	Grade 60	Steel			
29	-7.000	-10.00	1.272	Grade 60	Steel			
30	-2.333	-10.00	1.272	Grade 60	Steel			
31	2.333	-10.00	1.272	Grade 60	Steel			
32	7.000	-10.00	1.272	Grade 60	Steel			

Elastic Properties

Effective properties of the section, ignoring reinforcement.

Geometric Centroid	y	0.0in
	z	0.0in
Area		648.0in^2
Second Moments of Area	I_{yy}	69980.in^4
	I_{zz}	17500.in^4
	I_{yz}	0.0in^4
Principal Second Moments of Area	I_{uu}	69980.in^4
	I_{zz}	17500.in^4
	Angle	0.0°
Shear Area Factor	k_y	0.8333
	k_z	0.8333
Torsion Constant		48050.in^4

Bar	Y	Z	Diameter	Material	Type	Pre-stress Force	Pre-stress Strain	Appl. loads include/exclude pre-stress
	[in]	[in]	[in]			[kip]		

Section Modulus	Z_y	3888.in^3
	Z_z	1944.in^3
Plastic Modulus	Z_{py}	5832.in^3
	Z_{pz}	2916.in^3
Radius of Gyration	R_y	10.39in
	R_z	5.196in

Properties of gross section, including reinforcement.

Geometric Centroid	y	-4.688E-9in
	z	-0.07844in
EA		3.651E+6kip
EI	EI_{yy}	2.904E+6kip-ft^2
	EI_{zz}	702100.kip-ft^2
	EI_{yz}	0.001532kip-ft^2
Principal EI	EI_{uu}	2.904E+6kip-ft^2
	EI_{zz}	702100.kip-ft^2
	Angle	39.88E-9°

Section Material Properties

Type		Concrete
Name		6000 psi
Weight		Normal Weight
Density	ρ	145.0lb/ft^3
Cylinder Strength	f_c'	6.000kip/in^2
Tensile Strength	f_r	0.5809kip/in^2
Elastic Modulus (short term)	E	4463.kip/in^2
Poisson's Ratio	ν	0.2000
Coeff. Thermal Expansion	α	5.556E-6/°C
Maximum Strain		0.003000[-]
ULS Compression Curve		Rectangular
ULS Tension Curve		No-tension
SLS Compression Curve		Linear
SLS Tension Curve		Interpolated
Aggregate Size		0.0in

Reinforcement Properties

Name	Grade 60
f_y	60.00kip/in^2
Modulus	29000.kip/in^2
Maximum Strain	0.05000[-]
Stress/Strain Curve	Strain-hardening

Loading

Reference Point

All loading acts through the Reference Point.
All strain planes are defined relative to the Reference Point.

Definition		Geometric Centroid
Reference Point Coordinates	y	0.0in
	z	0.0in

Applied loads

Load Case	N	M_{yy}	M_{zz}
	[kip]	[kip-ft]	[kip-ft]
1	1235.	1305.	0.0

Section 1 Details

4.78% reinforcement in section 1 (Brace). Check this against code requirements.

Strength Analysis - Summary

Governing conditions are defined as:
A - reinforcing steel tension strain limit
B - concrete compression strain limit
Effective centroid is reported relative to the reference point.

Case	Eff. Centroid (y)	Eff. Centroid (z)	P	M	M_n	Strength Reduction (ϕ)	ϕM_n	$M/\phi M_n$	Governing Condition	Neutral Axis Angle	Neutral Axis Depth
			[kip]	[kip-ft]	[kip-ft]		[kip-ft]			[°]	[in]
1	-14.89E-9	-0.3772	1235.	1305.	2414.	0.6500	1569.	0.8316	B: Node 1	0.0	22.68

Strength Analysis - Details

Case	Moment Angle	Description	P	M	Warning
	[°]		[kip]	[kip-ft]	
	-180.0	Max. compressive strain	5004.	53.40	
	2.262E-6	Max. tensile strain	-1857.	58.37	
1	0.0	Axial strength at M	3772.	1305.	
		Balanced yield	1241.	2681.	
		Compressive strength at M=0	4954.	0.0	
		Bending strength at P=0	0.0	2093.	

Strain Planes at Nominal Strength

Case	Moment Angle [°]	Description	P [kip]	M [kip-ft]	Warning

Related to Reference Point

Case	Strain Plane	ε_{ax} [-]	κ_{yy} [/ft]	κ_{zz} [/ft]
1	Reinforcement	618.8E-6	0.001587	-60.74E-12
	User Creep/Shrinkage	0.0	0.0	0.0
	Total (Concrete)	618.8E-6	0.001587	-60.74E-12

Section Material Stresses/Strains at Nominal Strength

Case	Bar	Coordinates y [in]	z [in]	Strain [-]	Stress [kip/in²]	Notes
1	1	9.000	18.00	0.003000	5.100	
1	2	9.000	-18.00	-0.001762	0.0	
1	3	-9.000	-18.00	-0.001762	0.0	
1	4	-9.000	18.00	0.003000	5.100	

Reinforcement Stresses/Strains at Nominal Strength

Case	Bar	Coordinates y [in]	z [in]	Strain [-]	Stress [kip/in²]	Notes
1	1	-7.000	-14.00	-0.001233	-35.76	Grade 60
1	2	-2.333	-14.00	-0.001233	-35.76	Grade 60
1	3	2.333	-14.00	-0.001233	-35.76	Grade 60
1	4	7.000	-14.00	-0.001233	-35.76	Grade 60
1	5	-7.000	15.00	0.002603	60.00	Grade 60
1	6	-4.200	15.00	0.002603	60.00	Grade 60
1	7	-1.400	15.00	0.002603	60.00	Grade 60
1	8	1.400	15.00	0.002603	60.00	Grade 60
1	9	4.200	15.00	0.002603	60.00	Grade 60
1	10	7.000	15.00	0.002603	60.00	Grade 60
1	11	-7.000	-12.00	-968.6E-6	-28.09	Grade 60
1	12	-2.333	-12.00	-968.6E-6	-28.09	Grade 60
1	13	2.333	-12.00	-968.6E-6	-28.09	Grade 60
1	14	7.000	-12.00	-968.6E-6	-28.09	Grade 60
1	15	-7.000	12.00	0.002206	60.00	Grade 60
1	16	-4.200	12.00	0.002206	60.00	Grade 60
1	17	-1.400	12.00	0.002206	60.00	Grade 60
1	18	1.400	12.00	0.002206	60.00	Grade 60
1	19	4.200	12.00	0.002206	60.00	Grade 60
1	20	7.000	12.00	0.002206	60.00	Grade 60
1	21	-7.000	10.00	0.001942	56.31	Grade 60
1	22	7.000	10.00	0.001942	56.31	Grade 60
1	23	-7.000	8.000	0.001677	48.64	Grade 60
1	24	7.000	8.000	0.001677	48.64	Grade 60
1	25	-7.000	6.000	0.001413	40.96	Grade 60
1	26	7.000	6.000	0.001413	40.96	Grade 60
1	27	-7.000	4.000	0.001148	33.29	Grade 60
1	28	7.000	4.000	0.001148	33.29	Grade 60
1	29	-7.000	-10.00	-704.1E-6	-20.42	Grade 60
1	30	-2.333	-10.00	-704.1E-6	-20.42	Grade 60
1	31	2.333	-10.00	-704.1E-6	-20.42	Grade 60
1	32	7.000	-10.00	-704.1E-6	-20.42	Grade 60

Jacked Box Calculations

Fresno - Jacked Box Prelim Check

Design reference materials

-Principles of Foundation Engineering, 3rd Ed. - Braja M. Das
-LRFD Bridge Design Manual
-CALTRANS Design Manuals - Bridge Design
-Soil Mechanics - MIT 1969, Lambe & Whitman
-Seismic Structural Considerations for the Stem and Base of Retaining Walls Subjected to Earthquake Ground Motions - Ralph W. Strom and Robert M. Ebeling, 2005
-CHSRA Technical Memorandums

Unit Conversions

$kip := 1000lb$ $\quad pcf := \frac{lb}{ft^3}$ $\quad kcf := 1000pcf$

Soil Properties:

Soil Type - Silty – Sand

Soil Unit Weight : Dry - $\gamma d_{so} := 0.125kcf$

Soil Unit Weight : Wet - $\gamma w_{so} := 0.063kcf$

Effective Friction Angle - $\phi_{pr} := 35.0deg$

Wall Soil Interface Angle - $\delta_{wa} := 23.33 deg$

Soil Cohesion - $c_s := 0$

Concrete Properties:

Concrete Unit Weight - $\gamma_c := 0.150kcf$

Out-of-plane Analysis Length - $b_m := 12in$

Steel Yield Strength - $f_y := 60000\frac{lb}{in^2}$

Concrete Yield Strength - $f_c := 5000\frac{lb}{in^2}$

Jacked Box Characteristics:

Jacked Bx Out of Plan Node Spacing - $B_w := 30.0ft$

Jckd Bx Width - $L_w := 52.0ft$

Jckd Bx Wall Height - $H_w := 44.0ft$

Jckd Bx Soil Internal Angle - $\beta := 90.00deg$

Surcharge Angle At Backwall - $\alpha := 0.00deg$

Jckd Bx Slant Angle from Horizon - $\theta_{sw} := 0.00deg$

Soil Depth to Top of Box - $D_{tb} := 30.0ft$

Soil Depth to Bottom of Box - $D_{bb} := 74.0ft$

Water Depth Above Box - $Hw_{ab} := 5.0ft$

Water Unit Weight - $\gamma_w := 0.063kcf$

Design-Support Springs:

Soil Spring Calcs:

$\mu := 0.35$

Poisson's Ratio - Reference 5 Page 232

$\frac{L_w}{B_w} = 1.733$

$\beta_\alpha := 0.98$

$\beta_z := 2.18$

Horz. & Vert. Spring Constant Coefficients - Reference 5 Page 232

$E_s := 4000000 \frac{lb}{ft^2}$

Modulus for Silty-Sand

$G_s := \frac{E_s}{2 \cdot (1 + \mu)}$ $\qquad G_s = 1.481 \times 10^6 \frac{lb}{ft^2}$

Modulus of Rupture for Silty-Sand

$Khs := 2 \cdot (1 + \mu) \cdot G_s \cdot \beta_\alpha \cdot \sqrt{B_w \cdot L_w}$ $\qquad Khs = 1.548 \times 10^5 \cdot \frac{kip}{ft}$

Horz. Spring Constant for Subject Soil Conditions (formula - Reference 5 Page 231)

$Kvs := \frac{G_s}{(1 - \mu)} \cdot \beta_z \cdot \sqrt{B_w \cdot L_w}$ $\qquad Kvs = 1.962 \times 10^5 \cdot \frac{kip}{ft}$

Vert. Spring Constant for Subject Soil Conditions (formula - Reference 5 Page 231)

Design - Loads On Jacked Box

At-Rest Earth Pressure -

$K_o := 1 - \sin(\phi_{pr}) = 0.426$

Coulomb's Active Earth Pressure Calc's

Earth Pressure Coefficient -

$$\Gamma_a := \left[1 + \sqrt{\left(\frac{\sin(\phi_{pr} + \delta_{wa}) \cdot \sin(\phi_{pr} - \alpha)}{\sin(\beta - \delta_{wa}) \cdot \sin(\alpha + \beta)}\right)}\right]^2 \qquad \Gamma_a = 2.99$$

$$K_a := \frac{\left(\sin(\beta + \phi_{pr})\right)^2}{\sin(\beta)^2 \cdot \sin(\beta - \delta_{wa}) \cdot \Gamma_a} \qquad K_a = 0.244$$

MCE - Max. Considered Earthquake Components:

Earthquake Earth Pressure Coefficient (Mononobe-Okabe Methodology, Reference 5):

$PGA_{MCEh} := 0.250$ $\qquad k_{MCEh} := 0.200$

$PGA_{MCEv} := 0.190$ $\qquad k_{MCEv} := 0.095$

Horizontal and Vertical component of earthquake acceleration.

$\psi_{ae} := \operatorname{atan}\left(\frac{k_{MCEh}}{1 - k_{MCEv}}\right)$ $\qquad \psi_{ae} = 12.462 \deg$

$$\Gamma_{ae} := \left[1 + \sqrt{\left(\frac{\sin(\phi_{pr} + \delta_{wa})\cdot\sin(\phi_{pr} - \psi_{ae} - \alpha)}{\cos(\theta_{sw} + \delta_{wa} + \psi_{ae})\cdot\cos(\alpha - \theta_{sw})}\right)}\right]^2 \qquad \Gamma_{ae} = 2.671$$

$$MK_{ae} := \frac{\left(\cos(\phi_{pr} - \psi_{ae} - \theta_{sw})\right)^2}{\cos(\psi_{ae})\cdot\cos(\theta_{sw})^2\cdot\cos(\theta_{sw} + \psi_{ae} + \delta_{wa})\cdot\Gamma_{ae}} \qquad MK_{ae} = 0.403$$

$M\Delta K_{ae} := MK_{ae} - K_a = 0.159$

TM 2.9.10/6.10.13

MCE Soil (EH) Earthquake Load Unfactored

Horizontal Loading

$MP_{ae} := M\Delta K_{ae}\cdot\frac{1}{2}\cdot\left[\gamma d_{so}\cdot(1 - k_{MCEv})\right] H_w^2 = 17.401\cdot\frac{kip}{ft}$ $\qquad ED_{MCE} := MP_{ae}$ TM 2.9.10/6.10.13

Location from wall base

$zMP_{ae} := 0.65H_w$ $\qquad zMP_{ae} = 28.6ft$ $\qquad zED_{MCE} := zMP_{ae}$ TM 2.9.10/6.10.13

OBE - Operating Basic Earthquake Components:

Earthquake Earth Pressure Coefficient (Mononobe-Okabe Methodology, Reference 5):

$PGA_{OBEh} := 0.08$ $\qquad k_{OBEh} := 0.07$

$PGA_{OBEv} := 0.04$ $\qquad k_{OBEv} := 0.02$

Horizontal and Vertical component of earthquake acceleration.

$$\psi_{ae} := \operatorname{atan}\left(\frac{k_{OBEh}}{1 - k_{OBEv}}\right) \qquad \psi_{ae} = 4.086\,deg$$

$$\Gamma_{ae} := \left[1 + \sqrt{\left(\frac{\sin(\phi_{pr} + \delta_{wa})\cdot\sin(\phi_{pr} - \psi_{ae} - \alpha)}{\cos(\theta_{sw} + \delta_{wa} + \psi_{ae})\cdot\cos(\alpha - \theta_{sw})}\right)}\right]^2 \qquad \Gamma_{ae} = 2.896$$

$$OK_{ae} := \frac{\left(\cos(\phi_{pr} - \psi_{ae} - \theta_{sw})\right)^2}{\cos(\psi_{ae})\cdot\cos(\theta_{sw})^2\cdot\cos(\theta_{sw} + \psi_{ae} + \delta_{wa})\cdot\Gamma_{ae}} \qquad OK_{ae} = 0.287$$

$O\Delta K_{ae} := OK_{ae} - K_a = 0.043$

TM 2.9.10/6.10.13

OBE Soil (EH) Earthquake Load Unfactored

Horizontal Loading

$OP_{ae} := O\Delta K_{ae} \cdot \frac{1}{2} \cdot \left[\gamma d_{so} \cdot \left(1 - k_{OBEv}\right)\right] H_w^{\ 2} = 5.053 \frac{kip}{ft}$ $\quad ED_{OBE} := OP_{ae}$ $\quad$ TM 2.9.10/6.10.13

Location from wall base

$zOP_{ae} := 0.65 H_w$ $\quad zOP_{ae} = 28.6 ft$ $\quad zED_{OBE} := zOP_{ae}$ $\quad$ TM 2.9.10/6.10.13

Loads : 1-Foot Out-of-Plane Analysis

Vertical Loads - Top Slab

$EV := \left(\gamma d_{so} \cdot D_{bb} + \gamma w_{so} \cdot Hw_{ab}\right) \cdot 1ft = 9.565 \frac{kip}{ft}$ $\quad$ Soil Load

$WA_h := \gamma_w \cdot Hw_{ab} \cdot 1ft = 0.315 \frac{kip}{ft}$ $\quad$ Water Load

$DCabut_v$ => See attached spread sheet from geotechnical staff $\quad$ 1-foot analysis on top slab

Vertical Loads - Bottom Slab

$WA_b := -\gamma_w \cdot \left(Hw_{ab} + H_w\right) \cdot 1ft = -3.087 \frac{kip}{ft}$ $\quad$ Buoyancy Water Load

$DCv_{cb} := \gamma_c \cdot 2.5ft \cdot 1ft = 0.375 \frac{kip}{ft}$ $\quad$ Track concrete bed inside tunnel cell, vert.

$RRLL := 7.778 \frac{kip}{ft}$ $\quad \frac{70kip \cdot 2}{9ft \cdot 2} = 7.778 \frac{kip}{ft}$ $\quad$ Amtrak worst loading per track. TM 6.5.1.5

Two 70 kip axle loads at 9ft, divide by 2 to get per track per ft. No impact added per TM 6.5.2

Horizontal Loads - Walls

$EH_t := \left[K_o \cdot \gamma d_{so} \cdot \left(D_{tb} - Hw_{ab}\right) + K_o \cdot \gamma w_{so} \cdot Hw_{ab}\right] \cdot 1ft = 1.467 \frac{kip}{ft}$ $\quad$ Soil load at top of wall

$EH_{bt} := EH_t + K_a \cdot \gamma w_{so} \cdot H_w \cdot 1ft = 2.144 \frac{kip}{ft}$ $\quad$ Soil load at bottom of wall

$EHsur := K_o \cdot \left[\gamma d_{so} \cdot \left(D_{tb} - Hw_{ab}\right) + \gamma w_{so} \cdot Hw_{ab}\right] \cdot 1ft = 1.467 \frac{kip}{ft}$ $\quad$ Soil load due to surcharge

$EH_{abut} := 0.060 \frac{kip}{ft}$ $\quad$ Soil load due to abutment DL

$WA_t := \gamma_w \cdot Hw_{ab} \cdot 1ft = 0.315 \frac{kip}{ft}$ $\quad$ Water load at top of wall

$WA_{bt} := \gamma_w \cdot H_w \cdot 1ft = 2.772 \frac{kip}{ft}$ $\quad$ Water load at bottom of wall

$+ (0.25WA_h - 0.19WA_b) + EDEH_{MCE} * (zMP_{ae}) + 0.25[EH_{sur} + EH_{abut}]]$

Load Summaries & Load Combinations:

Dead Loads:
DCt : Dead load of primary tunnel components

DCv_{cb} : DL of concrete track bed inside tunnel cell

$DCabut_v$: DL induced force from abutment above tunnel cell

Live Loads:
RRLL : Railroad LL, worst case from Amtrak locomotives

Soil Loads:
EH : Soil load on side walls horizontal

EV : Soil load on top slab vertical

Water Loads:
WA_v : Water load on top slab vertical -Z direction

WA_h : Water load on side walls

WA_b : Water load buoyancy on bottom slab vertical +Z direction

Earthquake Loads:
$PGAh_{MCE}$: Peak horizontal ground acceleration at maximum considered earthquake load, (0.250 g)

$PGAv_{MCE}$: Peak vertical ground acceleration at maximum considered earthquake load, (0.190 g)

$PGAh_{OBE}$: Peak horizontal ground acceleration at operating basic earthquake load

$PGAv_{OBE}$: Peak vertical ground acceleration at operating basic earthquake load

$ED\text{-}DCh_t$: DL of primary tunnel components with horizontal EQ load induced

$ED\text{-}DCv_t$: DL of primary tunnel components with vertical EQ load induced

$ED\text{-}DCh_{cb}$: DL of concrete track bed with horizontal EQ load induced

$ED\text{-}DCv_{cb}$: DL of concrete track bed with vertical EQ load induced

ED-WA : Water load induced earthquake loads on the tunnel

$ED\text{-}EH_X$: Soil load induced surcharge and abutment DL loads on tunnel wall

$EDEH_{MCE}$ - Soil load induced earthquake load on tunnel wall

LRFD Load Combination Cases:

TM 6.7.1/TBL 6-2

TM 6.7.1 indicates using Service I, Strength I and Extreme Event III for the design analysis of the tunnel

Service I - (SI) : 1.0DC + 1.0EH + 1.0EV + 1.0LL + 1.0WA

$1.0[DCt + DCv_{cb} + DCabut_v] + 1.0[EH + EH_{sur} + EH_{abut}]\ 1.0EV + 1.0RRLL + 1.0[WA_v + WA_h - WA_b]$

Strength IA (max) - (SIA) : 1.25DC + 1.50EH + 1.30EV + 1.75LL + 1.60WA

$1.25[DCt + DCv_{cb} + DCabt_v] + 1.50[EH + EH_{sur} + EH_{abut}] + 1.30EV + 1.75RRLL + 1.60[WA_v + WA_h - WA_b]$

Strength IB (min) - (SIB) : 0.90DC + 0.90EH + 0.90EV + 1.75LL + 1.60WA

$0.90[DCt + DCv_{cb} + DCabt_v] + 0.90[EH + EH_{sur} + EH_{abut}] + 0.90EV + 1.75RRLL + 1.60[WA_v + WA_h - WA_b]$

Extreme Event III - (EE3) : 1.0DC + 1.0EH + 1.0EV + 1.0WA + 1.0ED

$1.0[DCt + DCv_{cb} + DCabt_v] + 1.0[EH + EH_{sur} + EH_{abut}] + 1.0EV + 1.0[WA_v + WA_h - WA_b]$
$+ 1.0[(0.25ED\text{-}DCh_t + 0.19ED\text{-}DCv_t + 0.25ED\text{-}DCh_{cb} + 0.19ED\text{-}DCv_{cb})$

Top Slab Check: Use two rows of tension steel - 2.5" cover 4" spacing

Tension Bar Size - 4#10 @ 8" spacing

Pseudo width / bar spacing / reduction factor

$b := 12$ $\quad sb_{ts} := 8$ $\quad \phi_v := 0.75$

Thickness / cover / depth of slab concrete

$t_{ts} := 60$ $\quad c_{ts} := 4.5$ $\quad d_{ts} := t_{ts} - c_{ts} = 55.5$

Area of steel / bar spacing / number per spacing

$As_{ts} := 1.27$ $\quad Sts_b := \dfrac{b}{sb_{ts}} = 1.5$ $\quad Nts_b := 8$

Shear Check -

Factored produced shear in slab, see attached calculations

$Vts_{IA} := 376.5\,kip$

$Vts_{cap} := \phi_v \cdot b \cdot d_{ts} \cdot 2 \cdot \sqrt{5000}$ $\quad Vts_{cap} = 7.064 \times 10^4$

$Vts_c := 731.9\,kip$

$\text{if}\left(Vts_c \geq Vts_{IA}, \text{"Shear Capacity Okay"}, \text{"Shear Capacity Failed"}\right) = \text{"Shear Capacity Okay"}$

Flexure Check -

Concrete reduction factor

$b_m := 12\,in$ $\quad f_y := 60000\dfrac{lb}{in^2}$ $\quad f_c := 5000\dfrac{lb}{in^2}$ $\quad d_m := d_{ts} \cdot in$ $\quad \rho_c := 0.90$

Total area of steel per ft of slab width

$As := As_{ts} \cdot Sts_b \cdot Nts_b \cdot in^2$

LRFD 5.7.2.2

$\beta_1 := 0.80$

LRFD 5.7.3.1.2-4

$cb_{ts} := \dfrac{As \cdot f_y}{0.85 \cdot f_c \cdot \beta_1 \cdot b_m} = 22.412\,in$ $\quad a := \beta_1 \cdot cb_{ts} = 17.929\,in$

Nominal allowable moment strength LRFD 5.7.2.2-1

$Mts_N := As \cdot f_y \cdot \left[d_m - \left(\dfrac{a}{2}\right)\right] = 3.546 \times 10^3 \cdot ft \cdot kip$

Factored allowable moment strength

$Mc_{ts} := \rho_c \cdot Mts_N$ $\quad Mc_{ts} = 3.191 \times 10^3 \cdot ft \cdot kip$

Max factored produced moment in slab

$Mts := 2294.8\,ft \cdot kip$

$\text{if}\left(Mc_{ts} \geq Mts, \text{"Moment Capacity Okay"}, \text{"Moment Capacity Failed"}\right) = \text{"Moment Capacity Okay"}$

Bottom Slab Check:

Bar Size - #11@ 8" spacing

$b := 12$ $\quad sb_{bs} := 8$ $\quad \phi_v := 0.75$ — Pseudo width / bar spacing / reduction factor

$t_{bs} := 60$ $\quad c_{bs} := 4.5$ $\quad d_{bs} := t_{bs} - c_{bs} = 55.5$ — Thickness / cover / depth of slab concrete

$As_{bs} := 1.56$ $\quad Sbs_b := \frac{b}{sb_{bs}} = 1.5$ $\quad Nbs_b := 1$ — Area of steel / bar spacing / number per spacing

Shear Check -

$Vbs_{IA} := 190.7 kip$ — Factored produced shear in slab, see attached calculations

$Vbs_{cap} := \phi_v \cdot b \cdot d_{bs} \cdot 2 \cdot \sqrt{5000}$ $\quad Vbs_{cap} = 7.064 \times 10^4$

$Vbs_c := 732.00 kip$

$\text{if}(Vbs_c \geq Vbs_{IA}, \text{"Shear Capacity Okay"}, \text{"Shear Capacity Failed"}) = \text{"Shear Capacity Okay"}$

Flexure Check -

$b_m = 12 \cdot in$ $\quad f_y = 6 \times 10^4 \cdot \frac{lb}{in^2}$ $\quad f_c = 5 \times 10^3 \cdot \frac{lb}{in^2}$ $\quad d_{m1} := d_{ts} \cdot in$ $\quad \rho_c := 0.90$ — Concrete reduction factor

$As_1 := As_{bs} \cdot Sbs_b \cdot Nbs_b \cdot in^2$ — Total area of steel per ft of slab width

$\beta_1 := 0.80$ — LRFD 5.7.2.2

$cb_{bs} := \frac{As_1 \cdot f_y}{0.85 \cdot f_c \cdot \beta_1 \cdot b_m} = 3.441 in$ $\quad a_{bs} := \beta_1 \cdot cb_{bs} = 2.753 in$ — LRFD 5.7.3.1.2-4

$Mbs_N := As_1 \cdot f_y \cdot \left[d_{m1} - \left(\frac{a_{bs}}{2}\right)\right] = 633.245 ft \cdot kip$ — Nominal allowable moment strength LRFD 5.7.2.2-1

$Mc_{bs} := \rho_c \cdot Mbs_N$ $\quad Mc_{bs} = 569.921 ft \cdot kip$ — Factored allowable moment strength

$Mbs := 416.29 ft \cdot kip$ — Max factored produced moment in slab

$\text{if}(Mc_{bs} \geq Mbs, \text{"Moment Capacity Okay"}, \text{"Moment Capacity Failed"}) = \text{"Moment Capacity Okay"}$

Wall Slab Check:

Bar Size - 4#10 @ 8" spacing

Pseudo width / bar spacing / reduction factor

$b := 12$ $\quad sb_{ws} := 8$ $\quad \phi_v := 0.75$

Thickness / cover / depth of slab concrete

$t_{ws} := 60$ $\quad c_{ws} := 4.5$ $\quad d_{ws} := t_{ws} - c_{ws} = 55.5$

Area of steel / bar spacing / number per spacing

$As_{ws} := 1.27$ $\quad Sws_b := \dfrac{b}{sb_{ws}} = 1.5$ $\quad Nws_b := 8$

Shear Check -

Factored produced shear in slab, see attached calculations

$Vws_{IA} := 376.5 kip$

$Vws_{cap} := \phi_v \cdot b \cdot d_{ws} \cdot 2 \cdot \sqrt{5000}$ $\quad Vws_{cap} = 7.064 \times 10^4$ $\quad Vws_c := 732.0 kip$

if$\left(Vws_c \geq Vws_{IA}, \text{"Shear Capacity Okay"}, \text{"Shear Capacity Failed"}\right) = \text{"Shear Capacity Okay"}$

Flexure Check -

$b_m = 12 \cdot in$ $\quad f_y = 6 \times 10^4 \cdot \dfrac{lb}{in^2}$ $\quad f_c = 5 \times 10^3 \cdot \dfrac{lb}{in^2}$ $\quad d_{m2} := d_{ws} \cdot in$ $\quad \rho_c := 0.90$

Concrete reduction factor

$As_2 := As_{ws} \cdot Sws_b \cdot Nws_b \cdot in^2$

Total area of steel per ft of slab width

$\beta_1 := 0.80$

LRFD 5.7.2.2

$cb_{ws} := \dfrac{As_2 \cdot f_y}{0.85 \cdot f_c \cdot \beta_1 \cdot b_m} = 22.412 in$ $\quad a_{ws} := \beta_1 \cdot cb_{ws} = 17.929 in$

LRFD 5.7.3.1.2-4

$Mws_N := As_2 \cdot f_y \cdot \left[d_{m2} - \left(\dfrac{a_{ws}}{2}\right)\right] = 3.546 \times 10^3 \cdot ft \cdot kip$

Nominal allowable moment strength LRFD 5.7.2.2-1

$Mc_{ws} := \rho_c \cdot Mws_N$ $\quad Mc_{ws} = 3.191 \times 10^3 \cdot ft \cdot kip$

Factored allowable moment strength

$Mws := 2294.8 ft \cdot kip$

Max factored produced moment in slab

if$\left(Mc_{ws} \geq Mws, \text{"Moment Capacity Okay"}, \text{"Moment Capacity Failed"}\right) = \text{"Moment Capacity Okay"}$

Axial Check -

$A_g := b_m \cdot t_{ws} \cdot 1in = 720 in^2$

Gross area of 1-ft of wall

$P_{n2} := 0.80 \left[0.85 f_c \cdot \left(A_g - As_2\right) + f_y \cdot As_2\right] = 3.128 \times 10^3 \cdot kip$

Nominal allowable axial strength

$Pc_{ws} := \rho_c \cdot P_{n2} = 2.815 \times 10^3 \cdot kip$

Factored axial load

$P_{ws} := 376.50 kip$

if$\left(Pc_{ws} \geq P_{ws}, \text{"Axial Capacity Okay"}, \text{"Axial Capacity Failed"}\right) = \text{"Axial Capacity Okay"}$

Dry Creek Culvert Calculations

Fresno - Dry Creek Canal Culvert Prelim Check

Design reference materials

-Principles of Foundation Engineering, 3rd Ed. - Braja M. Das
-LRFD Bridge Design Manual
-CALTRANS Design Manuals - Bridge Design
-Soil Mechanics - MIT 1969, Lambe & Whitman
-Seismic Structural Considerations for the Stem and Base of Retaining Walls Subjected to Earthquake Ground Motions - Ralph W. Strom and Robert M. Ebeling, 2005
-CHSRA Technical Memorandums

Unit Conversions

$kip := 1000 lb$ $\quad pcf := \frac{lb}{ft^3}$ $\quad kcf := 1000 pcf$

Wall Material Properties:

Soil Type - Silty – Sand

Concrete Unit Weight - $\gamma_c := 0.150 kcf$

Soil Unit Weight - $\gamma_{so} := 0.125 kcf$

Effective Friction Angle - $\phi_{pr} := 35.0 deg$

Wall Soil Interface Angle - $\delta_{wa} := 23.33 deg$

Soil Cohesion - $c_s := 0$

Canal Characteristics:

Canal Base Out of Plan Node Spacing - $B_w := 20.0 ft$

Canal Base Length - $L_w := 29.0 ft$

Canal Wall Height - $H_w := 8.60 ft$

Canal Wall Internal Angle - $\beta := 90.00 deg$

Surcharge Angle At Backwall - $\alpha := 0.00 deg$

Canal Wall Slant Angle from Horizon - $\theta_{sw} := 0.00 deg$

Design:

Soil Spring Calcs:

$\mu := 0.35$ Poisson's Ration - Reference 4 Page 232

$\frac{L_w}{B_w} = 1.45$

$\beta_\alpha := 0.96$

$\beta_z := 2.18$

Horz. & Vert. Spring Constant Coefficients - Reference 4 Page 232

$E_s := 4000000 \frac{lb}{ft^2}$ Modulus for Silty-Sand

$G_s := \frac{E_s}{2 \cdot (1 + \mu)}$ $\quad G_s = 1.481 \times 10^6 \frac{lb}{ft^2}$

Horz. Spring Constant for Subject Soil Conditions (formula - Reference 4 Page 231)

$Kvs := \frac{G_s}{(1 - \mu)} \cdot \beta_z \cdot \sqrt{B_w \cdot L_w}$ $\quad Kvs = 1.197 \times 10^5 \cdot \frac{kip}{ft}$

$Khs := 2 \cdot (1 + \mu) \cdot G_s \cdot \beta_\alpha \cdot \sqrt{B_w \cdot L_w}$

$Khs = 9.248 \times 10^4 \cdot \frac{kip}{ft}$

Coulomb's Active Earth Pressure Calc's

Earth Pressure Coefficient -

$$\Gamma_a := \left[1 + \sqrt{\left(\frac{\sin(\phi_{pr} + \delta_{wa})\cdot\sin(\phi_{pr} - \alpha)}{\sin(\beta - \delta_{wa})\cdot\sin(\alpha + \beta)}\right)}\right]^2 \qquad \Gamma_a = 2.99$$

$$K_a := \frac{\left(\sin(\beta + \phi_{pr})\right)^2}{\sin(\beta)^2\cdot\sin(\beta - \delta_{wa})\cdot\Gamma_a} \qquad K_a = 0.244$$

MCE - Max. Considered Earthquake Components:

Earthquake Earth Pressure Coefficient (Mononobe-Okabe Methodology, Reference 4):

$PGA_{MCEh} := 0.25($ $\qquad k_{MCEh} := 0.20($

$PGA_{MCEv} := 0.19($ $\qquad k_{MCEv} := 0.09\text{5}$

Horizontal and Vertical component of earthquake acceleration.

$$\psi_{ae} := \operatorname{atan}\left(\frac{k_{MCEh}}{1 - k_{MCEv}}\right) \qquad \psi_{ae} = 12.462\,deg$$

$$\Gamma_{ae} := \left[1 + \sqrt{\left(\frac{\sin(\phi_{pr} + \delta_{wa})\cdot\sin(\phi_{pr} - \psi_{ae} - \alpha)}{\cos(\theta_{sw} + \delta_{wa} + \psi_{ae})\cdot\cos(\alpha - \theta_{sw})}\right)}\right]^2 \qquad \Gamma_{ae} = 2.671$$

$$MK_{ae} := \frac{\left(\cos(\phi_{pr} - \psi_{ae} - \theta_{sw})\right)^2}{\cos(\psi_{ae})\cdot\cos(\theta_{sw})^2\cdot\cos(\theta_{sw} + \psi_{ae} + \delta_{wa})\cdot\Gamma_{ae}} \qquad MK_{ae} = 0.403$$

TM 2.9.10/6.10.13

$$M\Delta K_{ae} := MK_{ae} - K_a = 0.159$$

MCE Soil (EH) Earthquake Loads Unfactored

TM 2.9.10/6.10.13

Horizontal Loading

$$MP_{ae} := M\Delta K_{ae}\cdot\frac{1}{2}\cdot\left[\gamma_{so}\cdot(1 - k_{MCEv})\right]H_w^2 = 0.665\,\frac{kip}{ft} \qquad ED_{MCE} := MP_{ae}$$

Location from wall base

TM 2.9.10/6.10.13

$$zMP_{ae} := 0.65H_w \qquad zMP_{ae} = 5.59ft \qquad zED_{MCE} := zMP_{ae}$$

OBE - Operating Basic Earthquake Components:

Earthquake Earth Pressure Coefficient (Mononobe-Okabe Methodology, Reference 5):

$PGA_{OBEh} := 0.08$ $\qquad k_{OBEh} := 0.07$

$PGA_{OBEv} := 0.04$ $\qquad k_{OBEv} := 0.02$

Horizontal and Vertical Component of earthquake acceleration.

$$\psi_{ae} := \operatorname{atan}\left(\frac{k_{OBEh}}{1 - k_{OBEv}}\right) \qquad \psi_{ae} = 4.086\,deg$$

$$\Gamma_{ae} := \left[1 + \sqrt{\left(\frac{\sin(\phi_{pr} + \delta_{wa})\cdot\sin(\phi_{pr} - \psi_{ae} - \alpha)}{\cos(\theta_{sw} + \delta_{wa} + \psi_{ae})\cdot\cos(\alpha - \theta_{sw})}\right)}\right]^2 \qquad \Gamma_{ae} = 2.896$$

$$OK_{ae} := \frac{\left(\cos(\phi_{pr} - \psi_{ae} - \theta_{sw})\right)^2}{\cos(\psi_{ae})\cdot\cos(\theta_{sw})^2\cdot\cos(\theta_{sw} + \psi_{ae} + \delta_{wa})\cdot\Gamma_{ae}} \qquad OK_{ae} = 0.287$$

$$O\Delta K_{ae} := OK_{ae} - K_a = 0.043$$

TM 2.9.10/6.10.13

OBE Soil (EH) Earthquake Loads Unfactored

Horizontal Loading

$$OP_{ae} := O\Delta K_{ae}\cdot\frac{1}{2}\cdot\left[\gamma_{so}\cdot(1 - k_{OBEv})\right]H_w^2 = 0.193\,\frac{kip}{ft} \qquad ED_{OBE} := OP_{ae}$$

TM 2.9.10/6.10.13

Location from wall base

$$zOP_{ae} := 0.65H_w \qquad zOP_{ae} = 5.59ft \qquad zED_{OBE} := zOP_{ae}$$

TM 2.9.10/6.10.13

Top Slab Check:

Bar Size - #6 @ 12" spacing

$b := 12$ $sb_{ts} := 12$ $\phi_v := 0.75$

Pseudo width / bar spacing / reduction factor

$t_{ts} := 24$ $c_{ts} := 2.5$ $d_{ts} := t_{ts} - c_{ts} = 21.5$

Thickness / cover / depth of slab concrete

$As_{ts} := 0.44$ $Sts_b := \frac{b}{sb_{ts}} = 1$ $Nts_b := 1$

Area of steel / bar spacing / number per spacing

Shear Check -

Factored produced shear in slab, see attached calculations

$Vts_{IA} := 14.293 kip$

$Vts_{cap} := \phi_v \cdot b \cdot d_{ts} \cdot 2 \cdot \sqrt{5000}$ $Vts_{cap} = 2.737 \times 10^4$

$Vts_c := 27.37 kip$

if$\left(Vts_c \geq Vts_{IA}\right.$, "Shear Capacity Okay" , "Shear Capacity Failed"$\left.\right)$ = "Shear Capacity Okay"

Flexure Check -

$As := As_{ts} \cdot Sts_b \cdot Nts_b \cdot in^2$ $d_m := d_{ts} \cdot in$

Total area of steel per ft of slab width

$b_m := 12in$

$f_y := 60000 \frac{lb}{in^2}$

$f_c := 5000 \frac{lb}{in^2}$

$\rho_c := 0.90$

Concrete reduction factor

Max factored produced moment in slab

$Mts := 35.205\ ft \cdot kip$

$$Mts_N := As \cdot f_y \cdot \left[d_m - \left(\frac{As \cdot f_y}{1.7 \cdot f_c \cdot b_m}\right)\right]$$ $Mts_N = 46.731 ft \cdot kip$

Nominal allowable moment strength

$Mc_{ts} := \rho_c \cdot Mts_N$ $Mc_{ts} = 42.058 ft \cdot kip$

Factored allowable moment strength

if$\left(Mc_{ts} \geq Mts\right.$, "Moment Capacity Okay" , "Moment Capacity Failed"$\left.\right)$ = "Moment Capacity Okay"

Bottom Slab Check:

Bar Size - #6 @ 12" spacing

$b := 12$ $sb_{bs} := 12$ $\phi_v := 0.75$

Pseudo width / bar spacing / reduction factor

$t_{bs} := 18$ $c_{bs} := 2.5$ $d_{bs} := t_{bs} - c_{bs} = 15.5$

Thickness / cover / depth of slab concrete

$As_{bs} := 0.44$ $Sbs_b := \dfrac{b}{sb_{bs}} = 1$ $Nbs_b := 1$

Area of steel / bar spacing / number per spacing

Shear Check -

$Vbs_{IA} := 5.652\,kip$

Factored produced shear in slab, see attached calculations

$Vbs_{cap} := \phi_v \cdot b \cdot d_{bs} \cdot 2 \cdot \sqrt{5000}$ $Vbs_{cap} = 1.973 \times 10^4$

$Vbs_c := 19.73\,kip$

$\text{if}\left(Vbs_c \geq Vbs_{IA}, \text{"Shear Capacity Okay"}, \text{"Shear Capacity Failed"}\right) = \text{"Shear Capacity Okay"}$

Flexure Check -

$As_1 := As_{bs} \cdot Sbs_b \cdot Nbs_b \cdot in^2$ $d_{m1} := d_{bs} \cdot in$

Total area of steel per ft of slab width

$b_m = 12 \cdot in$

$f_y = 6 \times 10^4 \cdot \dfrac{lb}{in^2}$

$f_c = 5 \times 10^3 \cdot \dfrac{lb}{in^2}$

$\rho_c := 0.90$

Concrete reduction factor

$Mbs := 9.006\,ft \cdot kip$

Max factored produced moment in slab

$$Mbs_N := As_1 \cdot f_y \cdot \left[d_{m1} - \left(\frac{As_1 \cdot f_y}{1.7 \cdot f_c \cdot b_m}\right)\right]$$ $Mbs_N = 33.531\,ft \cdot kip$

Nominal allowable moment strength

$Mc_{bs} := \rho_c \cdot Mbs_N$ $Mc_{bs} = 30.178\,ft \cdot kip$

Factored allowable moment strength

$\text{if}\left(Mc_{bs} \geq Mbs, \text{"Moment Capacity Okay"}, \text{"Moment Capacity Failed"}\right) = \text{"Moment Capacity Okay"}$

Wall Slab Check:

Bar Size - #5 @ 12" spacing

Pseudo width / bar spacing / reduction factor

$b := 12$ $\quad sb_{ws} := 12$ $\quad \phi_v := 0.75$

Thickness / cover / depth of slab concrete

$t_{ws} := 18$ $\quad c_{ws} := 2.5$ $\quad d_{ws} := t_{ws} - c_{ws} = 15.5$

Area of steel / bar spacing / number per spacing

$As_{ws} := 0.31$ $\quad Sws_b := \frac{b}{sb_{ws}} = 1$ $\quad Nws_b := 1$

Shear Check -

Factored produced shear in slab, see attached calculations

$Vws_{IA} := 3.147 kip$

$Vws_{cap} := \phi_v \cdot b \cdot d_{ws} \cdot 2 \cdot \sqrt{5000}$ $\quad Vws_{cap} = 1.973 \times 10^4$

$Vws_c := 19.73 kip$

$\text{if}\left(Vws_c \geq Vws_{IA}, \text{"Shear Capacity Okay"}, \text{"Shear Capacity Failed"}\right) = \text{"Shear Capacity Okay"}$

Flexure Check -

Total area of steel per ft of slab width

$As_2 := As_{ws} \cdot Sws_b \cdot Nws_b \cdot in^2$

$b_m = 12 \cdot in$ $\quad d_{m2} := d_{ws} \cdot in$

$f_y = 6 \times 10^4 \cdot \frac{lb}{in^2}$

$f_c = 5 \times 10^3 \cdot \frac{lb}{in^2}$

Concrete reduction factor

$\rho_c := 0.90$

Max factored produced moment in slab

$Mws := 12.736 ft \cdot kip$

Nominal allowable Moment strength

$Mws_N := As_2 \cdot f_y \cdot \left[d_{m2} - \left(\frac{As_2 \cdot f_y}{1.7 \cdot f_c \cdot b_m}\right)\right]$ $\quad Mws_N = 23.742 ft \cdot kip$

Factored allowable moment strength

$Mc_{ws} := \rho_c \cdot Mws_N$ $\quad Mc_{ws} = 21.368 ft \cdot kip$

$\text{if}\left(Mc_{ws} \geq Mws, \text{"Moment Capacity Okay"}, \text{"Moment Capacity Failed"}\right) = \text{"Moment Capacity Okay"}$

Appendix C – Fresno Street Bridge

SEISMIC ANALYSIS AND DESIGN PLAN

General Classification

As this structure directly supports the HST track it is designated a **Primary Structure** in accordance with TM 2.10.4.

Importance Classification

The structure lies on the main route south of Fresno Station. Therefore in accordance with TM 2.10.4 cl 6.5.1.2 it is proposed to be designated an Important Structure.

Technical Classification

The structure does not conform to the requirements of a Standard Structure. Neither does it possess any of the features that might class it as a complex structure. Therefore in accordance with TM 2.10.4 cl 6.5.1.3 it is proposed to be designated a **Non-Standard** Structure.

Analysis Approach

The bridge is located on a section of route at the approach to Fresno Station where there are 4 tracks. The deck is divided into two identical halves in order to limit the amount of transverse thermal expansion that is experienced by the substructure.

One half of the superstructure will be modeled using SAP 2000. As the structure is essentially a single simply supported span it will be modeled as a single line element representing the full cross section for one half.

Using SAP 2000 a modal frequency analysis will be carried out to confirm that the frequency limits of TM 2.10.10 are satisfied.

Following this, response spectrum analysis will be conducted for the OBE and MCE events. Track-structure interaction will also be investigated.

For track structure interaction analysis Oasys GSA will be used. A line model representing two rails will be constructed. It will be supported on non-linear springs representing the track connection clips and connected either to ground supports or to a further line beam representing the bridge deck which will itself be supported. It is not thought necessary to model the foundations or piles for this analysis.

Seismic Response Mechanism

The structure articulation is arranged so that the superstructure is supported on a number of disc bearing pads. Each deck will be supported by four "fixed" bearings at one end and four "guided" bearings at the other. Lateral expansion/contraction due to temperature will be partially restrained by the columns in bending. Each bearing pad is located at the top of a stub column that is designed to provide a limited plastic hinge capacity to protect the foundations in the MCE event.

At the OBE level event the structure is intended to behave elastically and the bearing pads will be sized to accommodate the expected movements associated with the OBE event.

At the MCE level event it is expected that the stub columns will deform inelastically. Forces will be limited by their plastic hinge capacity. The foundations will be designed to carry the factored plastic hinge forces without becoming inelastic themselves.

As a long stop against excessive movement, the decks will be restrained transversely by the provision of shear keys between decks and at the ends of the abutments. Longitudinally,

movements will be constrained by earth pressures from the ends of the deck, which bear directly against the structure backfill.

Bridge Configuration

Fresno Street Bridge is a 100-foot long single-span High Speed Train bridge. The superstructure is 100 feet wide and consists of two identical decks of 50-feet wide. The superstructure is a voided slab with rectangular voids. A small gap separates the two decks.

As it spans over an underpass with retaining walls designed by others, the bridge foundation is independent of the retaining walls and piles are placed through the retained soil. The superstructure is placed on top of six-foot tall, 3'x3' square concrete columns with bearings. These columns sit on top of pile caps to ensure easy inspection and plastic hinge formation under extreme event. The soil bearing wall is designed to extend the superstructure down to the top of the pile cap. It is expected to take the longitudinal shear in the superstructure during extreme event.

The foundation system consists of 145-foot long, 3' diameter concrete piles under a rigid pile cap. Soil spring stiffnesses were developed by the geotechnical engineer and shown in Figure 1.

The stiffness values in Figure 1 do not include values below a depth of 60 feet but these are included in the analysis model. It should also be noted that the stiffness becomes non-uniform for displacements that exceed approximately 0.01 in for the soils from 0 to approximately 20 feet. This has been accommodated in the analysis by specifying non-linear springs to represent the soils where displacements may enter the non-linear zone.

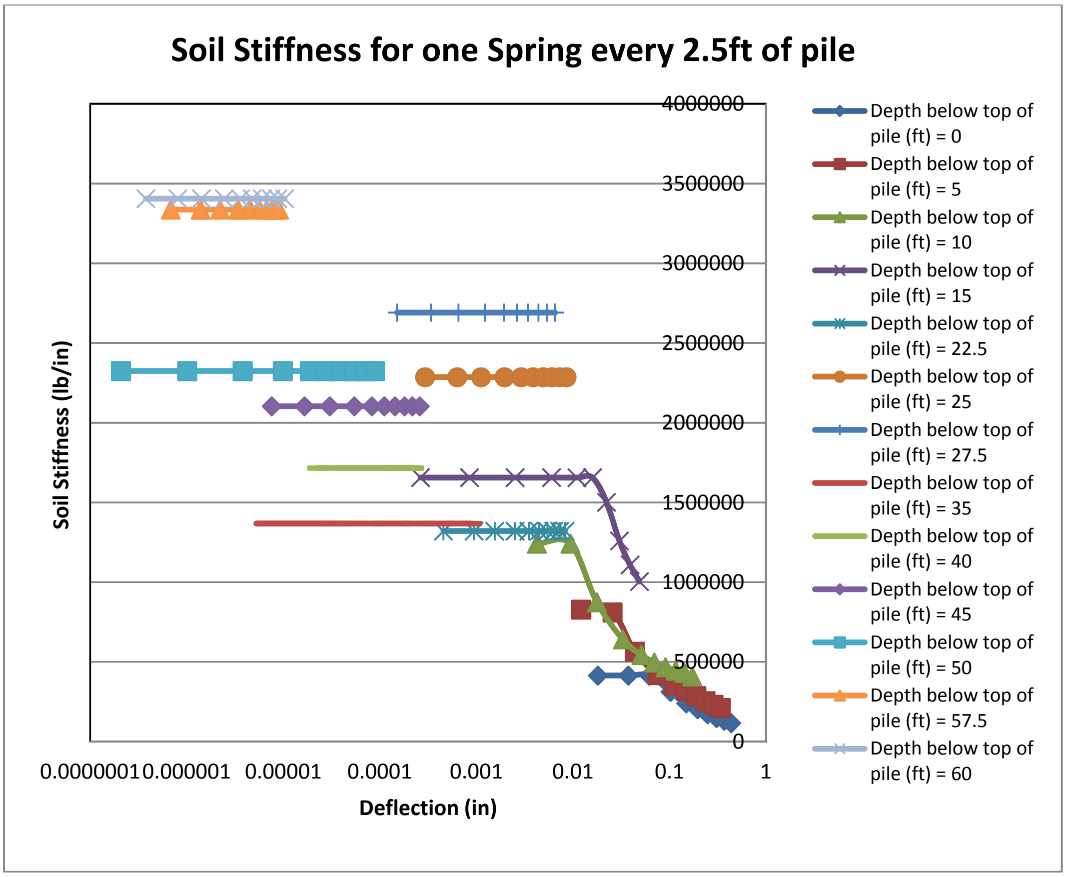


Figure 1 Stiffness used for Soil Springs in SAP2000 model

Seismic Design

The seismic design has been carried out according to Tech Memo 2.10.4: Seismic Design Criteria. Caltrans Seismic Design Criteria were also used to establish demand and capacity of displacement ductility.

Displacement Checks: $\Delta_D \leq \Delta_C$

Demand Ductility Checks: $\mu_D \leq 5$ for Multiple Column Bents

Capacity Ductility Checks: $\mu_C \geq 3$

The structure is designed to be elastic under the Operational Basis Earthquake (OBE), and the columns take all the seismic demand longitudinally and transversely. Under Maximum Considered Earthquake (MCE), the longitudinal force is resisted by passive soil pressure in the soil bearing wall connected to the superstructure. Transverse seismic force is taken by the columns. The shear keys act as a final restraint for extreme displacements.

SAP Model Analysis

The analysis was carried out using a SAP2000 model with the bridge module. Only half of the complete structure was modeled as the two parts are identical. The superstructure is modeled as a line element, connected by rigid links to individual columns. The columns are then connected via rigid links to the top of piles, to represent the rigidity of the pile caps.

Lateral soil springs representing the stiffness of Figure 1 were attached in both horizontal directions at every 5 feet of pile length. For springs less than 20 feet below the top of pile, these springs are modeled as non-linear as the above graph shows non-linear behavior. At small displacements and depths greater than 20 feet the analysis results were checked and show that displacements under the static load cases and response spectrum analyses are within the linear region and so linear springs have been used to represent these soil properties.

Vertical soil springs are also applied in the model to represent skin friction effects. This provides a more representative model of the behavior of the structure for vertical loads and modes of vibration. The spring stiffness used was derived from a comparison of pile reaction and vertical displacement of the pile cap. Springs with equal stiffness were then used uniformly throughout the length of the pile.

Structure self weight is automatically calculated by SAP2000 based on standard material densities.

The superimposed dead load has been calculated at 9.4 klf as used in the standard structure calculations.

Live load has been taken to be the Modified Cooper E-50 train set and this has been used in the calculation of impact loads. Braking and traction forces were applied to calculate track-structure interaction.

Seismic checks were carried out using the Response Spectrum as indicated in the Geotechnical Design Memorandum using SAP's dynamic response spectra analyses feature.

Note that non-linear static analysis was used for all load cases except seismic OBE and MCE, which were analyzed using only linear spring properties.

For load cases that combine seismic and other loads, linear addition was used.

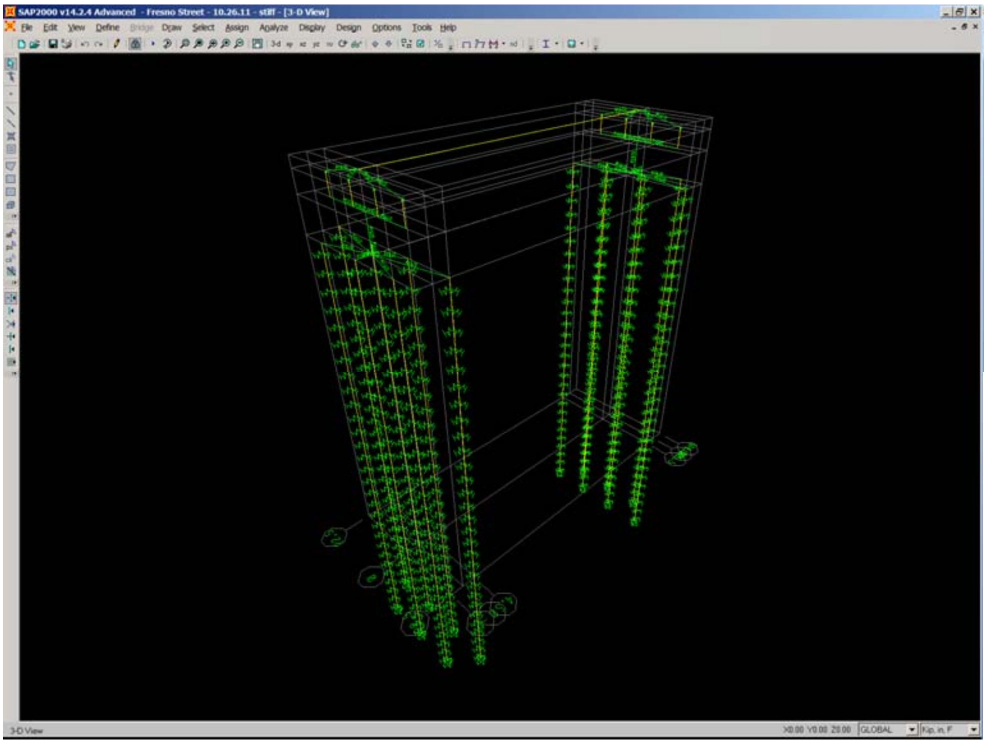


Figure 2 View of SAP Model

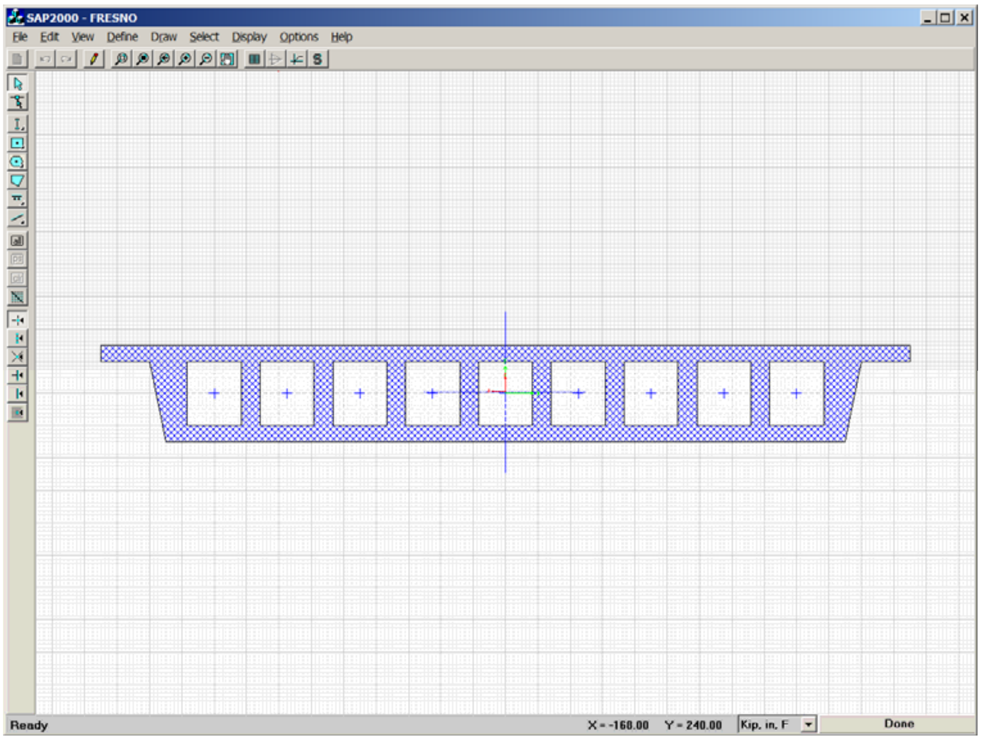


Figure 3 Cross Section of Deck Element with rectangular voids

Frequency Analysis Results

Frequency (Hz)	Stiff Model	Soft Model
Longitudinal	4.06	3.90
Transverse	5.89	5.56
Vertical	3.30	3.21
Torsional	10.71	10.08

Two models with different assumptions of mass and stiffness were used to envelope the frequency analysis. The calculation for acceptable bounds for the frequency results is attached as a calculation sheet.

The results show that all the frequency requirements of TM 2.10.10 are met.

Track Structure Interaction Analysis

As required by TM 2.10.10, the line model of the rail extends off the bridge at both ends for a length of 230-feet (L+130). The total length of track modeled is therefore 560-feet. At each end of this model a grounded spring has been added with a longitudinal stiffness of 10500 kip/ft to represent the continuity of the rails beyond the model.

The rail section used for modeling purposes is the AREMA 141RE section which has a cross sectional area of 13.8 sq-in. It has been assumed that track clips are located at 2-ft centers along the rails. Which means that each modeled clip (which represents 2 clips) can develop a maximum limit load of 9 kips for loaded condition, and 4 kips for unloaded condition.

For simplicity, substructure components are not included in this track-structure interaction model. The deck is supported at the two ends with simply supported connections at the bottom of deck. If substructure is included, the relative longitudinal displacement and deck end rotation due to vertical loads would be less than the current model where the neutral axis of the bridge is pinned. For 30% design, this simplification of track-structure interaction is conservative and justified.

3 load cases have been studied to understand the interaction between the rail and the structure.

1. Train Braking on approach to the structure at the point just before reaching the structure, i.e. Braking force applied over 230-feet. Train traction applied on bridge only (100 feet).

2. Train Braking on approach to the structure where the front of the train has passed onto the structure over its full length and is just about to leave at the remote end, i.e. Braking force applied over 330-feet. Train traction applied on bridge only (100 feet).

3. Train Braking having passed over the structure by a distance of 230-feet i.e. braking force applied to full length of rail, 560-feet. Train traction applied on bridge only (100 feet).

This analysis considers both the braking force (1.37 kip/ft) and traction force (2.26 kip/ft). It is possible that traction and braking can occur at the same time and this would be the worst scenario. Line load for braking and traction forces are applied directly to the rail elements. At the same time a vertical UDL of 10kip/ft, representing the Cooper E50 live load plus vertical impact load, is applied for each track where braking and traction forces are applied. These three cases represent Group 4 load case as required in TM 2.10.10 Section 6.6.2. This is also a conservative model as both braking and traction forces are applied to the same track for simplicity of modeling, but since the results show deformation well within limits, this model is acceptable for a 30% design.

Group 5 load case requires one track of live, braking and traction forces instead of both tracks in Group 4, with the addition of OBE seismic loads. Since inputs for non-linear time history analysis are not yet provided in technical memorandum, elastic response spectrum analysis is used for relative joint displacement between bridge and abutments. According to SAP2000 analysis, relative joint displacement under OBE loadcase is 0.11" and is much smaller than maximum allowed value of 0.50" as required under TM 2.10.10 cl. 6.6.3. Therefore the following analysis will only focus on Group 4 load case.

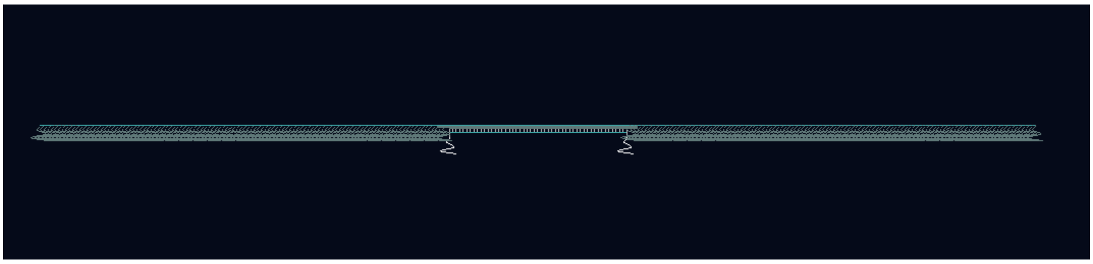


Figure 4 GSA Model of Track & Structure

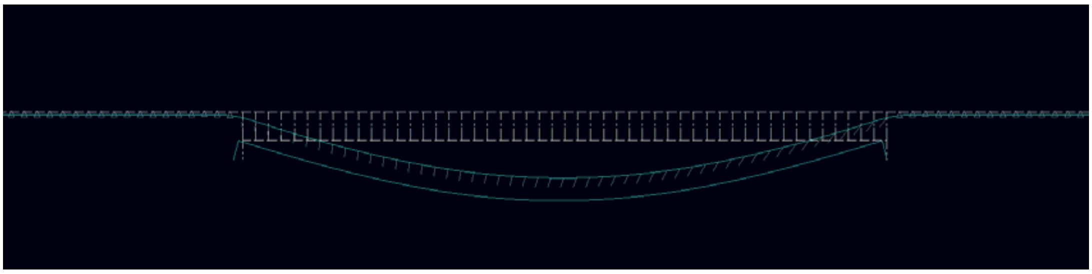
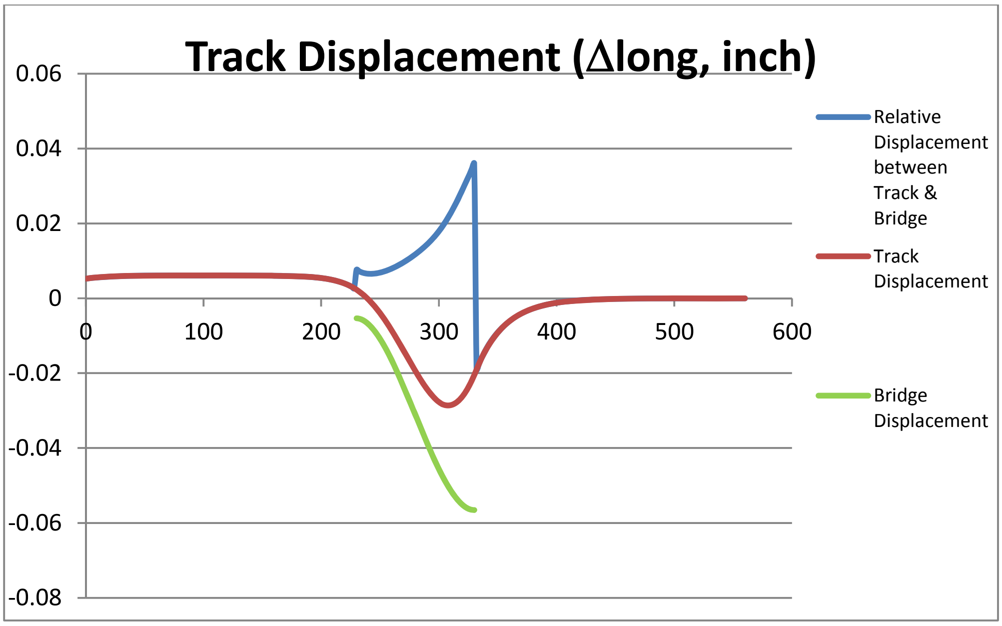
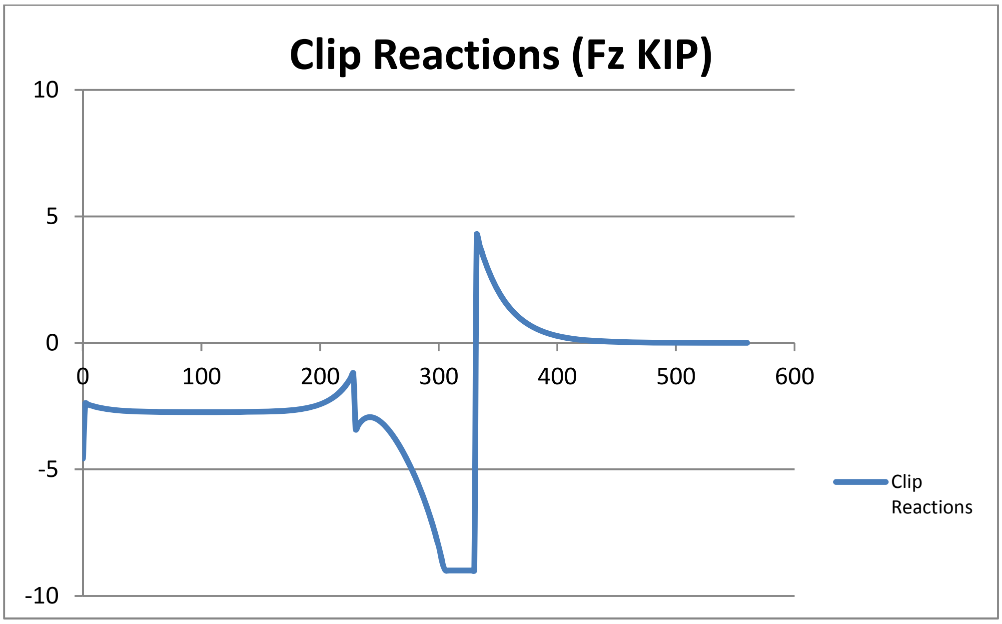


Figure 5 Example of Analysis Output under Case 3

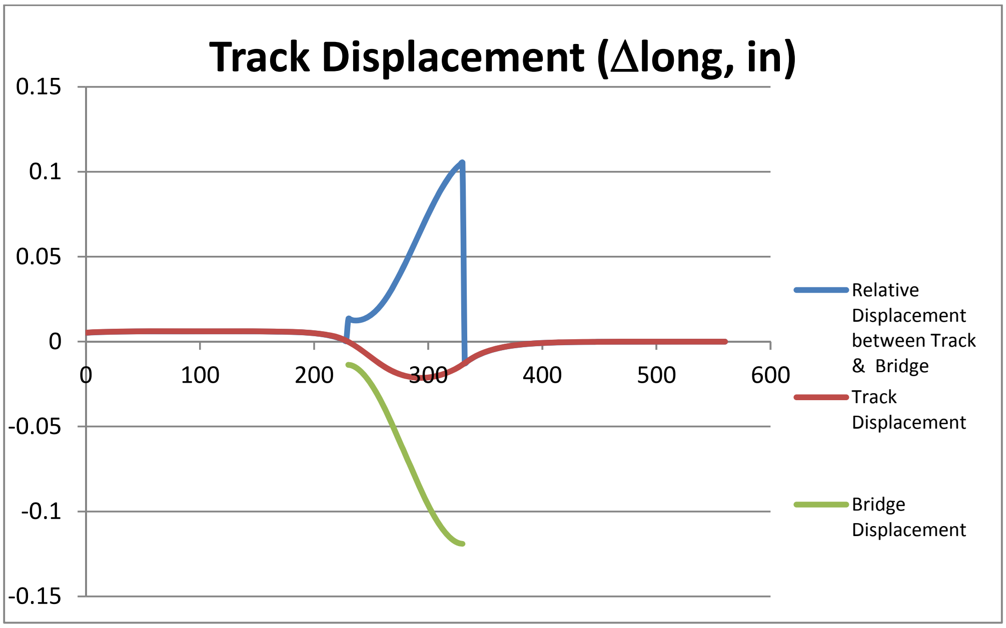
The results of the analysis are summarized in the table below.

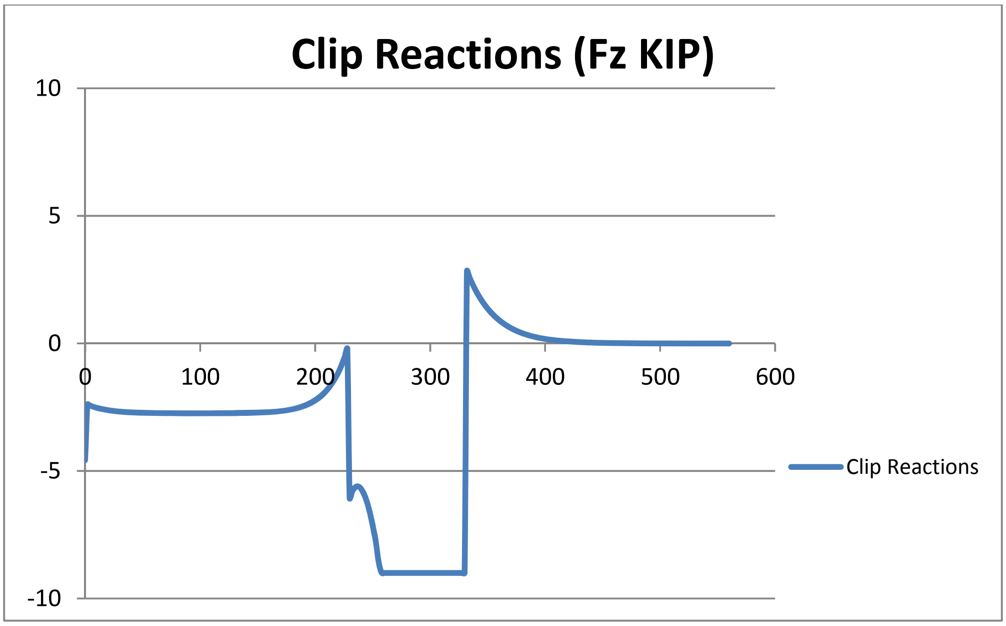
Case 1



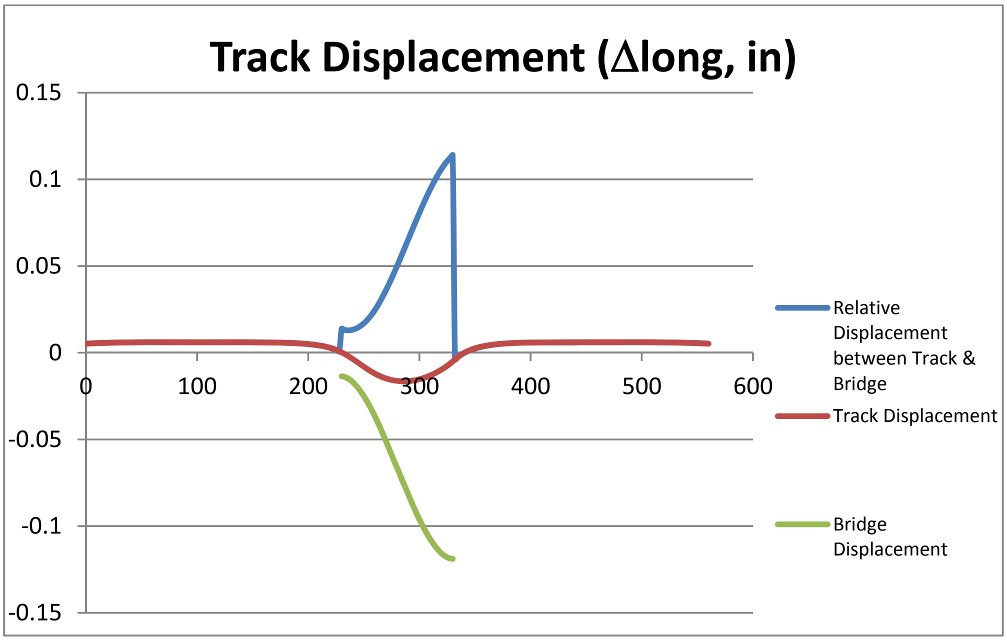


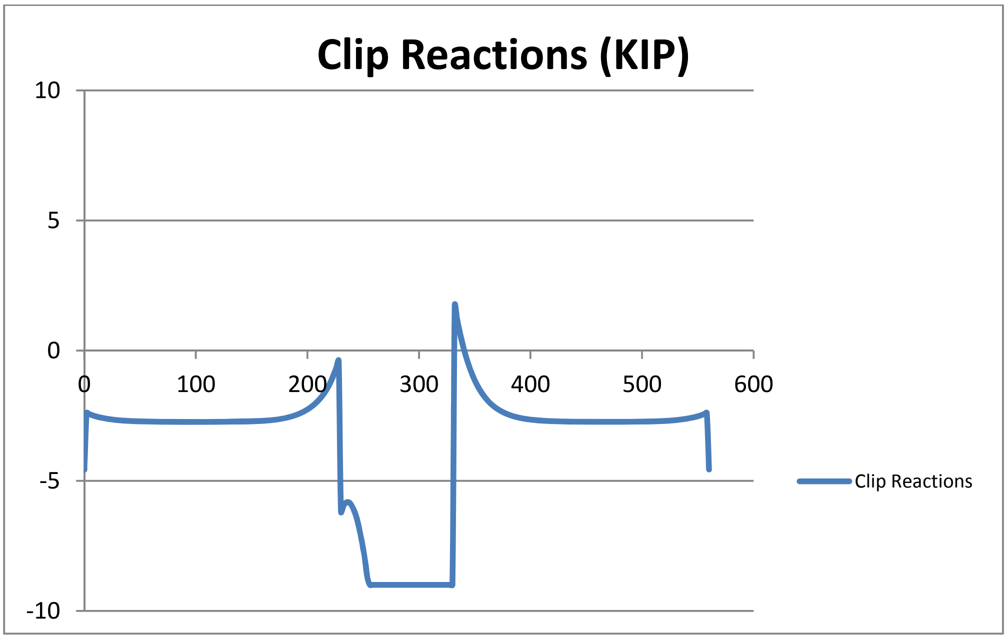
Case 2


Track Displacement (Δlong, in)
0.15
0.1
0.05
0
-0.05
-0.1
-0.15
0
100
200
300
400
500
600
Relative Displacement between Track & Bridge
Track Displacement
Bridge Displacement


Clip Reactions (Fz KIP)
10
5
0
-5
-10
0
100
200
300
400
500
600
Clip Reactions

Case 3


Track Displacement (Δlong, in)
0.15
0.1
0.05
0
-0.05
-0.1
-0.15
0
100
200
300
400
500
600
Relative Displacement between Track & Bridge
Track Displacement
Bridge Displacement


Clip Reactions (KIP)
10
5
0
-5
-10
0
100
200
300
400
500
600
Clip Reactions

These graphs from the three analyses cases show a number of key features.

Case 1

The track displacement of around 0.006” in the left abutment is caused by braking force. At the bridge deck, bridge displacement peaks at 0.053” to the left.

The clips at the left abutment deform to the right, and those at the right abutment deform to the left. The clips on the bridge deform to the right.

Case 2

The track displacement of around 0.006” in the left abutment is caused by braking force as in Case 1. At the bridge deck, bridge displacement peaks at 0.119” to the left. The increase in these values compared to Case 1 is due to the additional vertical force on the bridge.

The clips on the left bridge end deform to the right, and those at the right abutment deform to the left. The clips on the bridge deform to the right.

Case 3

This case is broadly similar to Case 2. There are minor differences due to the track clips to the right of the bridge being stiffer because they are loaded. The track displaces at the right abutment due to the braking force being applied here.

Results

According to clause 6.5.3, vertical static deflection in span due to LLRM and Impact shall not exceed L/750=1.6” for both tracks loaded. Deflection is 0.56” and is well within limit. For single track load, limit is L/2200=0.55”. Deflection of one track of LLRM and Impact is 0.28” and is well within limit.

According to clause 6.6.3, rotation between abutment and deck due to LLRM+I (two tracks) is 0.0025 rad, which is smaller than 0.003 rad as required.

Displacement demand and limit at expansion joints are summarized in the following table according to TM 2.10.10 clause 6.6.3.

Loadcase (TM 2.10.10 cl 6.6.3)	Displacement	Displacement Limit	Vertical Deck Deflection	Deflection Limit
Group 4 – Case 1	0.057”	0.5”	0.029”	0.08”
Group 4 – Case 2	0.119”	0.5”	0.059”	0.08”
Group 4 – Case 3	0.119”	0.5”	0.059”	0.08”

Seismic Analysis Results

MCE Level, Transverse Direction

The capacity ductility is very high at 18. Under the MCE event, demand displacement is about 0.8 times the yield displacement, and thus the demand ductility is acceptable under the current seismic design criteria. The demand ductility is much smaller than the capacity ductility so the design is considered to be adequate. Excessive movement will be limited by shear key.

Please see attached XTRACT Analysis Report and ductility calculations.

MCE Level, Longitudinal Direction

The capacity ductility is very high at 18. Under the MCE event, demand displacement is 1.9 times the yield displacement, and thus the demand ductility of 1.9 is acceptable under the current seismic design criteria. The demand ductility is much smaller than the capacity ductility so the design is considered to be adequate. Excessive movement will be limited by the soil bearing wall at the end of deck.

Please see attached XTRACT Analysis Report and ductility calculations.

OBE Level, Both Directions

The columns are designed to be elastic under OBE level. Shear design was checked using the checking module within SAP. The results are considered adequate with a reasonable level of reinforcement. The SAP2000 results are attached.

Superstructure Post-Tensioning Analysis

The deck section was checked against AASHTO LRFD requirements as documented in the following calculations.

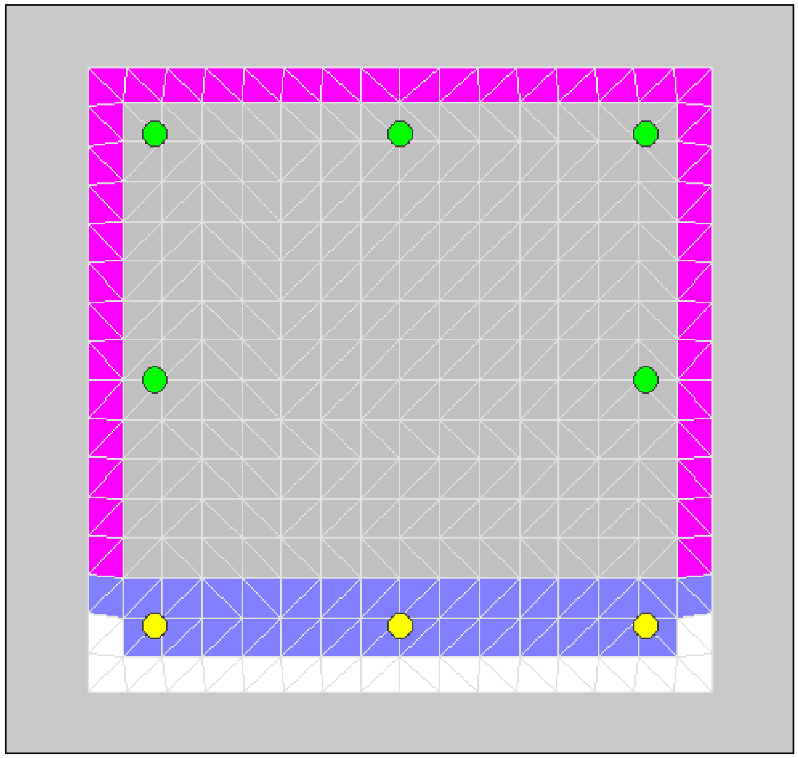
XTRACT Analysis Report

ARUP - San Francisco, CA
Arup
12/6/2011
CHSTP
Fresno Street Pushover

Section Name: Section1
Loading Name: M-Phi
Analysis Type: Moment Curvature

Section Details:

X Centroid:	.1345E-15 in
Y Centroid:	.7657E-16 in
Section Area:	1296 in^2

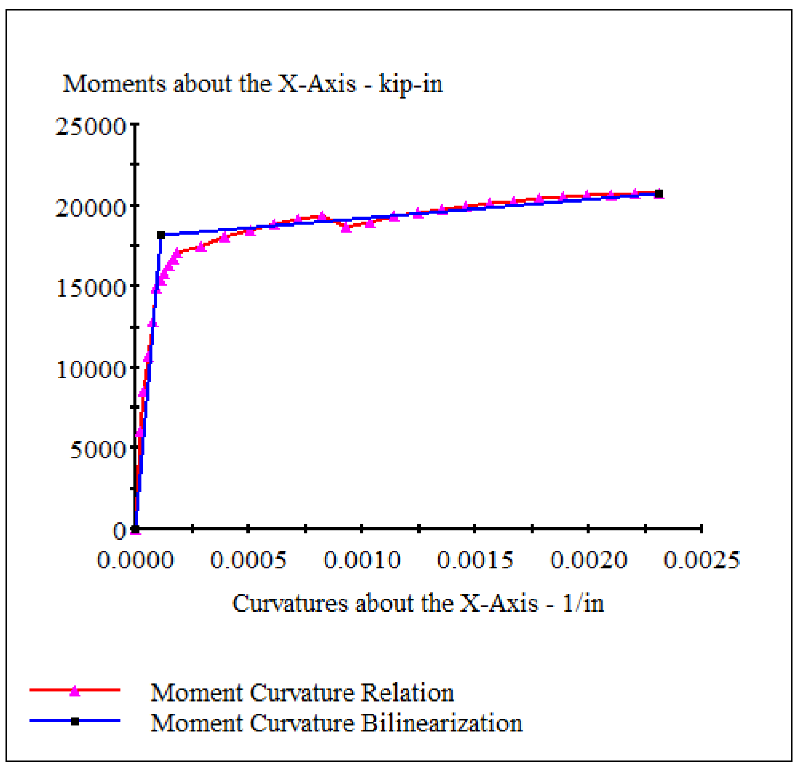


Loading Details:

Constant Load - P:	400.0 kips
Incrementing Loads:	Mxx Only
Number of Points:	30
Analysis Strategy:	Displacement Control

Analysis Results:

Failing Material:	Steel1
Failure Strain:	60.00E-3 Tension
Curvature at Initial Load:	.6151E-23 1/in
Curvature at First Yield:	92.58E-6 1/in
Ultimate Curvature:	2.309E-3 1/in
Moment at First Yield:	14.96E+3 kip-in
Ultimate Moment:	20.74E+3 kip-in
Centroid Strain at Yield:	.7570E-3 Ten
Centroid Strain at Ultimate:	27.28E-3 Ten
N.A. at First Yield:	8.177 in
N.A. at Ultimate:	11.81 in
Energy per Length:	43.77 kips
Effective Yield Curvature:	.1125E-3 1/in
Effective Yield Moment:	18.18E+3 kip-in
Over Strength Factor:	1.141
EI Effective:	1.62E+8 kip-in^2
Yield EI Effective:	1.167E+6 kip-in^2
Bilinear Harding Slope:	.7221 %
Curvature Ductility:	20.53



Comments:

User Comments

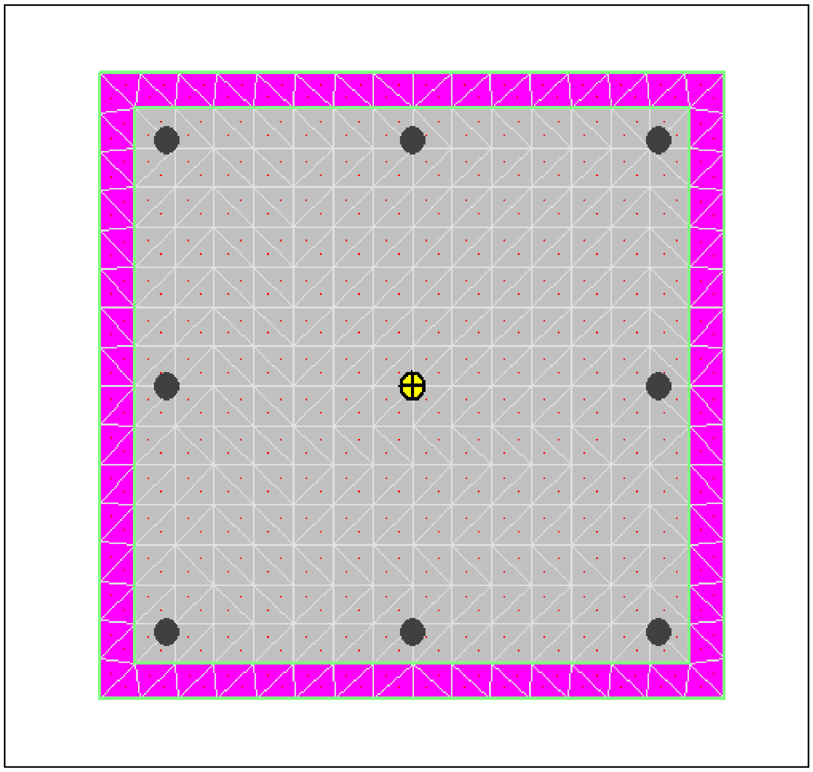
XTRACT Section Report

ARUP - San Francisco, CA
Arup
12/6/2011
CHSTP
Fresno Street Pushover
Page __ of __

Section Name: Section1

Section Details:

X Centroid:	.1345E-15 in
Y Centroid:	.7657E-16 in
Section Area:	1296 in^2
EI gross about X:	6.63E+8 kip-in^2
EI gross about Y:	6.63E+8 kip-in^2
I trans (Confined1) about X:	150.1E+3 in^4
I trans (Confined1) about Y:	150.1E+3 in^4
Reinforcing Bar Area:	12.50 in^2
Percent Longitudinal Steel:	.9642 %
Overall Width:	36.00 in
Overall Height:	36.00 in
Number of Fibers:	512
Number of Bars:	8
Number of Materials:	3



Material Types and Names:

Unconfined Concrete:	Unconfined1
Confined Concrete:	Confined1
Strain Hardening Steel:	Steel1

XTRACT Material Report

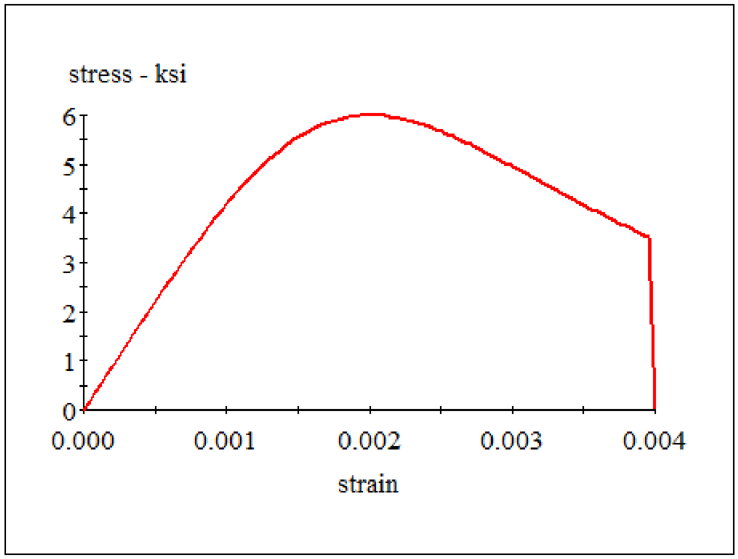
ARUP - San Francisco, CA
Arup
12/6/2011
CHSTP
Fresno Street Pushover
Page __ of __

Material Name: Unconfined1

Material Type: Unconfined Concrete

Input Parameters:

Tension Strength:	0 ksi
28 Day Strength:	6.000 ksi
Post Crushing Strength:	0 ksi
Tension Strain Capacity:	0 Ten
Spalling Strain:	4.000E-3 Comp
Failure Strain:	1.0000 Comp
Elastic Modulus:	4415 ksi
Secant Modulus:	3000 ksi



Model Details:

For Strain - $\varepsilon < 2 \cdot \varepsilon_t$ $\quad fc = 0$

For Strain - $\varepsilon < 0$ $\quad fc = \varepsilon \cdot Ec$

For Strain - $\varepsilon < \varepsilon_{cu}$ $\quad fc = \dfrac{f'_c \cdot x \cdot r}{r - 1 + x^r}$

For Strain - $\varepsilon < \varepsilon_{sp}$ $\quad fc = f_{cu} + \left(f_{cp} - f_{cu}\right) \cdot \dfrac{\left(\varepsilon - \varepsilon_{cu}\right)}{\left(\varepsilon_{sp} - \varepsilon_{cu}\right)}$

$$x = \frac{\varepsilon}{\varepsilon_{cc}}$$

$$r = \frac{Ec}{Ec - E_{sec}}$$

$$E_{sec} = \frac{f'_c}{\varepsilon_{cc}}$$

ε = Concrete Strain

fc = Concrete Stress

Ec = Elastic Modulus

E_{sec} = Secant Modulus

ε_t = Tension Strain Capacity

ε_{cu} = Ultimate Concrete Strain

ε_{cc} = Strain at Peak Stress = .002

ε_{sp} = Spalling Strain

f'_c = 28 Day Compressive Strength

f_{cu} = Stress at ε_{cu}

f_{cp} = Post Spalling Strength

Material Color States:

- Tension strain after tension capacity
- Tension strain before tension capacity
- Initial state
- Compression before crushing strain
- Compression before end of spalling
- Compression after spalling

Reference:

Mander, J.B., Priestley, M. J. N., "Observed Stress-Strain Behavior of Confined Concrete", Journal of Structural Engineering, ASCE, Vol. 114, No. 8, August 1988, pp. 1827-1849

XTRACT Material Report

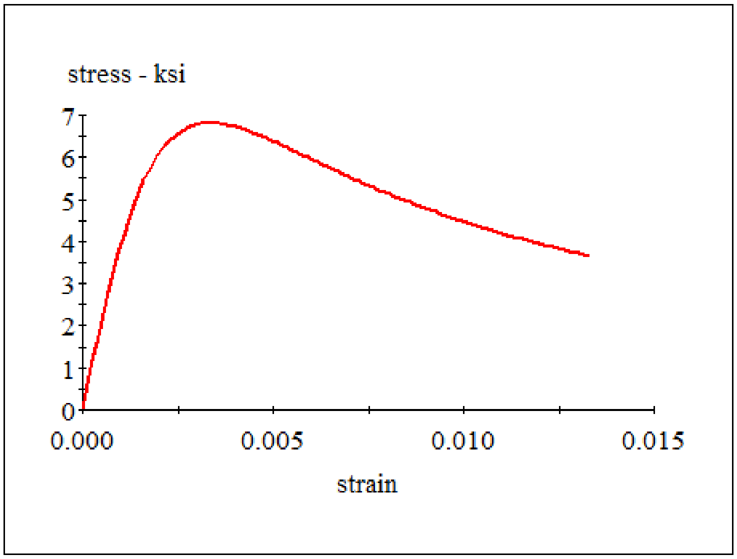
ARUP - San Francisco, CA
Arup
12/6/2011
CHSTP
Fresno Street Pushover
Page __ of __

Material Name: Confined1

Material Type: Confined Concrete

Input Parameters:

Tension Strength:	0 ksi
28 Day Strength:	6.000 ksi
Confined Concrete Strength:	6.814 ksi
Tension Strain Capacity:	0 Ten
Strain at Peak Stress:	3.357E-3
Crushing Strain:	13.30E-3 Comp
Elastic Modulus:	4415 ksi
Secant Modulus:	2030 ksi



Model Details:

For Strain - $\varepsilon < 2 \cdot \varepsilon_t$	$fc = 0$
For Strain - $\varepsilon < 0$	$fc = \varepsilon \cdot Ec$
For Strain - $\varepsilon < \varepsilon_{cu}$	$fc = \dfrac{f_{cc} \cdot x \cdot r}{r - 1 + x^r}$

$$x = \frac{\varepsilon}{\varepsilon_{cc}}$$

$$\varepsilon_{cc} = .002 \cdot \left[1 + 5 \cdot \left(\frac{f_{cc}}{f_c} - 1\right)\right]$$

$$r = \frac{Ec}{Ec - E_{sec}}$$

$$E_{sec} = \frac{f_{cc}}{\varepsilon_{cc}}$$

ε = Concrete Strain

fc = Concrete Stress

Ec = Elastic Modulus

ε_t = Tension Strain Capacity

ε_{cu} = Ultimate Concrete Strain

ε_{cc} = Strain at Peak Stress

f_c = 28 Day Compressive Strength

f_{cc} = Confined Concrete Strength

Material Color States:

- Tension strain after tension capacity
- Tension strain before tension capacity
- Initial state
- Compression before crushing strain

Reference:

Mander, J.B., Priestley, M. J. N., "Observed Stress-Strain Behavior of Confined Concrete", Journal of Structural Engineering, ASCE, Vol. 114, No. 8, August 1988, pp. 1827-1849

XTRACT Material Report

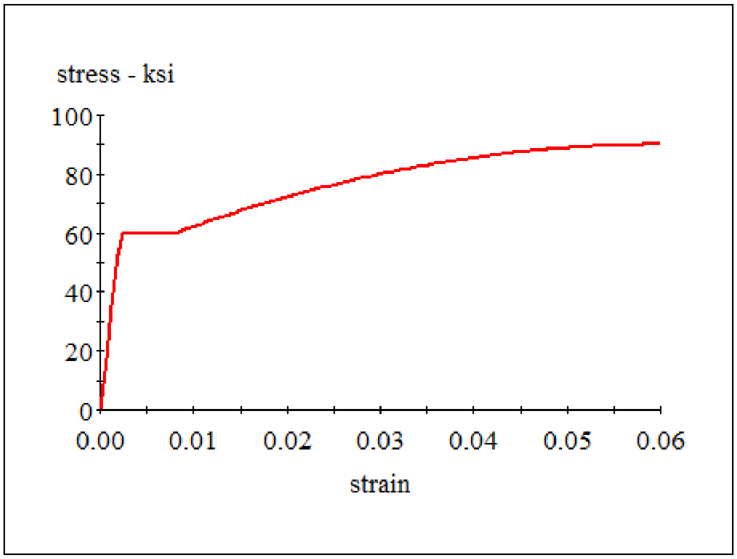
ARUP - San Francisco, CA
Arup
12/6/2011
CHSTP
Fresno Street Pushover

Material Name: Steel1

Material Type: Strain Hardening Steel

Input Parameters:

Yield Stress:	60.00 ksi
Fracture Stress:	90.00 ksi
Yield Strain:	2.069E-3
Strain at Strain Hardening:	8.000E-3
Failure Strain:	60.00E-3
Elastic Modulus:	29.00E+3 ksi
Additional Information:	Symetric Tension and Comp.



Model Details:

For Strain - $\varepsilon < \varepsilon_y$ $\quad fs = E \cdot \varepsilon$

For Strain - $\varepsilon < \varepsilon_{sh}$ $\quad fs = f_y$

For Strain - $\varepsilon < \varepsilon_{su}$ $\quad fs = f_u - (f_u - f_y) \cdot \left(\frac{\varepsilon_{su} - \varepsilon}{\varepsilon_{su} - \varepsilon_{sh}} \right)^2$

ε = Steel Strain

fs = Steel Stress

f_y = Yield Stress

f_u = Fracture Stress

ε_y = Yield Strain

ε_{sh} = Strain at Strain Hardening

ε_{su} = Failure Strain

E = Elastic Modulus

Material Color States:

- Tension force after onset of strain hardening
- Tension force after yield
- Initial state
- Compression force after yield
- Compression force after onset of strain hardening

Consider a slice of the deck as a rectangular section (with voids) based on a 9ft width.

Section Dimensions

Deck slab width $b := 9 \cdot ft$

Slab Thickness $h := 6 \cdot ft$

Rebar Cover $c_{nom} := 3 \cdot in$

Material Properties

Prestress strand strength $f_{pu} := 270 \cdot \frac{kip}{in^2}$ $\quad f_{py} := 0.85 \cdot f_{pu} = 229.5 \cdot \frac{kip}{in^2}$

Concrete strength Grade $f_c := 6000 \cdot \frac{lbf}{in^2}$

Concrete Short Term Elastic Modulus $E_c := 5000 \cdot \frac{kip}{in^2}$ $\quad$ Long term $E_{cl} := \frac{E_c}{2}$

Section Properties

Void Width $d_v := 40 \cdot in$ $\quad$ Number of voids $n_v := \frac{b}{\left(d_v + 14 \cdot in\right)}$ $\quad n_v = 2$

Void Height $d_h := 48 \cdot in$

Gross Area $A_g := b \cdot h - n_v \cdot d_v \cdot d_h$ $\quad A_g = 27.333 \cdot ft^2$

Symmetrical Section $h_{NA} := \frac{h}{2}$ $\quad y_t := h_{NA}$ $\quad y_b := h_{NA}$

$$I_{NA} := \frac{b \cdot h^3}{12} - \frac{n_v \cdot d_h^{\,3} \cdot d_v}{12} \qquad I_{NA} = 2.622 \times 10^6 \cdot in^4$$

Section moduli $Z_{top} := \frac{I_{NA}}{y_t}$ $\quad Z_{bot} := \frac{I_{NA}}{y_b}$

$Z_{top} = 7.283 \times 10^4 \cdot in^3$

$Z_{bot} = 7.283 \times 10^4 \cdot in^3$

Prestress Allowance

Assume 4 Ducts each with 19 strands per web.
That is 8 ducts altogether in a 9ft width (2 webs).

Area of strand $A_s := 0.217 \cdot in^2$ $\quad 19 \cdot A_s = 4.123 \cdot in^2$

$A_p := 8 \cdot 19 \cdot A_s = 32.984 \cdot in^2$ $\quad dia_d := 4 \cdot in$ $\quad A_{duct} := \pi \frac{dia_d^{\,2}}{4} = 12.566 \cdot in^2$

Assume ducts achieve maximum eccentricity at mid span. Assume that the ducts spread out so that all can achieve maximum eccentricity.

$e := \frac{h}{2} - c_{nom} - 1 \cdot in - 0.5 \cdot dia_d$ $\quad e = 30 \cdot in$

Prestress force (80% UTS) $\quad P := A_p \cdot 0.8 \cdot f_{pu}$ $\quad P = 7.125 \times 10^3 \cdot kip$

Weight of Section

Concrete density $\gamma_c := 160 \cdot \frac{lb}{ft^3}$

Span $L := 100 \cdot ft$

From SAP2000 model output, $M_{sw} := \frac{26692 kip \cdot ft}{\left(\frac{50ft}{b}\right)}$ $M_{sw} = 4.805 \times 10^3 \cdot kip \cdot ft$

Superimposed Load

From SAP2000 model output, $M_{DW} := \frac{11533 kip \cdot ft}{\left(\frac{50ft}{b}\right)}$ $M_{DW} = 2.076 \times 10^3 \cdot kip \cdot ft$

Live Load

Allow for tractor unit of Cooper E50 on deck. A single 10ft width can carry a single track only. Ignore load sharing / spread.

From SAP2000 model output, one track:

$M_{LL} := \frac{15975 kip \cdot ft}{2}$ $M_{LL} = 7.988 \times 10^3 \cdot kip \cdot ft$

Importance

Importance classifications

$\eta_I := 1.05$ Critical structure for HST

$\eta_R := 1.05$ Non - redundant

$\eta_D := 1.05$ Non - ductile

$\eta := \eta_I \cdot \eta_R \cdot \eta_D$ $\eta = 1.158$

Impact Effect for LLRR Live Load

$IM := \frac{225}{\left(\frac{L}{ft}\right)^{0.5}}$ $IM = 22.5$

Stress Checks at Transfer

Transfer Load Factors

Dead Load $\gamma_{DC} := 1.0$

Super Dead Load $\gamma_{DW} := 1.0$

Locked IN Force (Prestress) $\gamma_{EL} := 1.0$

$M_T := \eta \cdot \left(M_{sw} \cdot \gamma_{DC} + M_{DW} \cdot \gamma_{DW}\right)$ $M_T = 7.965 \times 10^3 \cdot kip \cdot ft$

Assume Short term losses of prestress are $\text{loss} := 10 \cdot \%$

Transfer Stresses

$$\sigma_{top} := \frac{(1-\text{loss}) \cdot P}{A_g} - \frac{(1-\text{loss}) \cdot P \cdot e}{Z_{top}} + \frac{M_T}{Z_{top}} \qquad \sigma_{top} = 0.3 \cdot \frac{kip}{in^2}$$

$$\sigma_{bot} := \frac{(1-\text{loss}) \cdot P}{A_g} + \frac{(1-\text{loss}) \cdot P \cdot e}{Z_{bot}} - \frac{M_T}{Z_{bot}} \qquad \sigma_{bot} = 2.958 \cdot \frac{kip}{in^2}$$

$$\sigma_{comp.transfer\_limit} := 0.6 \cdot f_c = 3.6 \cdot \frac{kip}{in^2} \qquad \sigma_{tens.transfer\_limit} := -0.24 \cdot \left(\frac{f_c}{\frac{kip}{in^2}} \right)^{0.5} \frac{kip}{in^2} = -0.588 \cdot \frac{kip}{in^2}$$

Service Load Factors

Dead Load $\gamma_{DC} := 1.0$

Super Dead Load $\gamma_{DW} := 1.0$

Live Load $\gamma_{LL} := 1.0$

Locked IN Force (Prestress) $\gamma_{EL} := 1.0$

Assume long term prestress loss is $\text{loss} := 25 \cdot \%$

Service Stresses

$$M_S := \eta \cdot \left[M_{sw} \cdot \gamma_{DC} + M_{LL} \cdot \gamma_{LL} \cdot \left(1 + \frac{IM}{100} \right) + M_{DW} \cdot \gamma_{DW} \right] = 1.929 \times 10^4 \cdot kip \cdot ft$$

$$\sigma_{top} := \frac{(1-\text{loss}) \cdot P}{A_g} - \frac{(1-\text{loss}) \cdot P \cdot e}{Z_{top}} + \frac{M_S}{Z_{top}} \qquad \sigma_{top} = 2.335 \cdot \frac{kip}{in^2}$$

$$\sigma_{bot} := \frac{(1-\text{loss}) \cdot P}{A_g} + \frac{(1-\text{loss}) \cdot P \cdot e}{Z_{bot}} - \frac{M_S}{Z_{bot}} \qquad \sigma_{bot} = 0.38 \cdot \frac{kip}{in^2}$$

$$\sigma_{comp.servce\_limit} := 0.45 \cdot f_c = 2.7 \cdot \frac{kip}{in^2} \qquad \sigma_{tens.service\_limit} := -0.19 \cdot \left(\frac{f_c}{\frac{kip}{in^2}} \right)^{0.5} \frac{kip}{in^2} = -0.465 \cdot \frac{kip}{in^2}$$

Elastic shortening of deck over time

$$\Delta := \frac{P \cdot L}{A_g \cdot E_{cl}} \qquad \Delta = 0.869 \cdot in$$

Abutment piles will be pulled together by this amount. The majority of this will occur during stressing.

TABLE: Concrete Design 1 - Column Summary Data - AASHTO Concrete 07														
Frame	DesignSect	DesignType	DesignOpt	Status	Location	PMMCombo	PMMArea	PMMRatio	VMajCombo	VMajRebar	VMinCombo	VMinRebar	ErrMsg	WarnMsg
Text	Text	Text	Text	Text	in	Text	in2	Unitless	Text	in2/in	Text	in2/in	Text	Text
8	3' PILE	Column	Design	No Messages	0	DCON11	17.2311		DCON12	0.255	DCON12	0.255	No Messages	No Messages
8	3' PILE	Column	Design	No Messages	15	DCON11	14.0863		DCON12	0.255	DCON12	0.255	No Messages	No Messages
8	3' PILE	Column	Design	No Messages	30	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
10	3' PILE	Column	Design	No Messages	0	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
10	3' PILE	Column	Design	No Messages	15	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
10	3' PILE	Column	Design	No Messages	30	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
14	3' PILE	Column	Design	No Messages	0	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
14	3' PILE	Column	Design	No Messages	30	DCON11	14.1131		DCON12	0.255	DCON12	0.255	No Messages	No Messages
14	3' PILE	Column	Design	No Messages	60	DCON11	16.7266		DCON12	0.255	DCON12	0.255	No Messages	No Messages
15	3' PILE	Column	Design	No Messages	0	DCON11	14.8154		DCON12	0.255	DCON12	0.255	No Messages	No Messages
15	3' PILE	Column	Design	No Messages	30	DCON11	14.2143		DCON12	0.255	DCON12	0.255	No Messages	No Messages
15	3' PILE	Column	Design	No Messages	60	DCON11	13.5775		DCON12	0.255	DCON12	0.255	No Messages	No Messages
16	3' PILE	Column	Design	No Messages	0	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
16	3' PILE	Column	Design	No Messages	30	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
16	3' PILE	Column	Design	No Messages	60	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
17	3' PILE	Column	Design	No Messages	0	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
17	3' PILE	Column	Design	No Messages	30	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
17	3' PILE	Column	Design	No Messages	60	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
18	3' PILE	Column	Design	No Messages	0	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
18	3' PILE	Column	Design	No Messages	30	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
18	3' PILE	Column	Design	No Messages	60	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
19	3' PILE	Column	Design	No Messages	0	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
19	3' PILE	Column	Design	No Messages	30	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
19	3' PILE	Column	Design	No Messages	60	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
20	3' PILE	Column	Design	No Messages	0	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
20	3' PILE	Column	Design	No Messages	30	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
20	3' PILE	Column	Design	No Messages	60	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
21	3' PILE	Column	Design	No Messages	0	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
21	3' PILE	Column	Design	No Messages	30	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
21	3' PILE	Column	Design	No Messages	60	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
22	3' PILE	Column	Design	No Messages	0	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
22	3' PILE	Column	Design	No Messages	30	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
22	3' PILE	Column	Design	No Messages	60	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
23	3' PILE	Column	Design	No Messages	0	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
23	3' PILE	Column	Design	No Messages	30	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
23	3' PILE	Column	Design	No Messages	60	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
24	3' PILE	Column	Design	No Messages	0	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
24	3' PILE	Column	Design	No Messages	30	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
24	3' PILE	Column	Design	No Messages	60	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
25	3' PILE	Column	Design	No Messages	0	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
25	3' PILE	Column	Design	No Messages	30	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
25	3' PILE	Column	Design	No Messages	60	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
26	3' PILE	Column	Design	No Messages	0	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
26	3' PILE	Column	Design	No Messages	30	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
26	3' PILE	Column	Design	No Messages	60	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
27	3' PILE	Column	Design	No Messages	0	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
27	3' PILE	Column	Design	No Messages	30	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
27	3' PILE	Column	Design	No Messages	60	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
28	3' PILE	Column	Design	No Messages	0	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
28	3' PILE	Column	Design	No Messages	30	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
28	3' PILE	Column	Design	No Messages	60	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
29	3' PILE	Column	Design	No Messages	0	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
29	3' PILE	Column	Design	No Messages	30	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
29	3' PILE	Column	Design	No Messages	60	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
30	3' PILE	Column	Design	No Messages	0	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
30	3' PILE	Column	Design	No Messages	30	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
30	3' PILE	Column	Design	No Messages	60	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
31	3' PILE	Column	Design	No Messages	0	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
31	3' PILE	Column	Design	No Messages	30	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
31	3' PILE	Column	Design	No Messages	60	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
32	3' PILE	Column	Design	No Messages	0	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
32	3' PILE	Column	Design	No Messages	30	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
32	3' PILE	Column	Design	No Messages	60	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
52	3' PILE	Column	Design	No Messages	0	DCON11	17.1782		DCON12	0.255	DCON12	0.255	No Messages	No Messages
52	3' PILE	Column	Design	No Messages	15	DCON11	14.0112		DCON12	0.255	DCON12	0.255	No Messages	No Messages
52	3' PILE	Column	Design	No Messages	30	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
53	3' PILE	Column	Design	No Messages	0	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
53	3' PILE	Column	Design	No Messages	15	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
53	3' PILE	Column	Design	No Messages	30	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
54	3' PILE	Column	Design	No Messages	0	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
54	3' PILE	Column	Design	No Messages	30	DCON11	14.0375		DCON12	0.255	DCON12	0.255	No Messages	No Messages
54	3' PILE	Column	Design	No Messages	60	DCON11	16.6702		DCON12	0.255	DCON12	0.255	No Messages	No Messages
55	3' PILE	Column	Design	No Messages	0	DCON11	14.7559		DCON12	0.255	DCON12	0.255	No Messages	No Messages
55	3' PILE	Column	Design	No Messages	30	DCON11	14.1518		DCON12	0.255	DCON12	0.255	No Messages	No Messages
55	3' PILE	Column	Design	No Messages	60	DCON11	13.5117		DCON12	0.255	DCON12	0.255	No Messages	No Messages
56	3' PILE	Column	Design	No Messages	0	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
56	3' PILE	Column	Design	No Messages	30	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
56	3' PILE	Column	Design	No Messages	60	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
57	3' PILE	Column	Design	No Messages	0	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
57	3' PILE	Column	Design	No Messages	30	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages
57	3' PILE	Column	Design	No Messages	60	DCON12	12.96		DCON12	0.255	DCON12	0.255	No Messages	No Messages

58	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
58	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
58	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
59	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
59	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
59	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
60	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
60	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
60	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
61	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
61	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
61	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
62	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
62	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
62	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
63	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
63	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
63	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
64	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
64	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
64	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
65	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
65	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
65	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
66	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
66	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
66	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
67	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
67	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
67	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
68	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
68	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
68	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
69	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
69	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
69	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
70	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
70	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
70	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
71	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
71	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
71	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
72	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
72	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
72	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
73	3' PILE	Column	Design	No Messages	0	DCON11	17.2399	DCON12	0.255	DCON12	0.255	No Messages	No Messages
73	3' PILE	Column	Design	No Messages	15	DCON11	14.0937	DCON12	0.255	DCON12	0.255	No Messages	No Messages
73	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
74	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
74	3' PILE	Column	Design	No Messages	15	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
74	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
75	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
75	3' PILE	Column	Design	No Messages	30	DCON11	14.1074	DCON12	0.255	DCON12	0.255	No Messages	No Messages
75	3' PILE	Column	Design	No Messages	60	DCON11	16.7259	DCON12	0.255	DCON12	0.255	No Messages	No Messages
76	3' PILE	Column	Design	No Messages	0	DCON11	14.8148	DCON12	0.255	DCON12	0.255	No Messages	No Messages
76	3' PILE	Column	Design	No Messages	30	DCON11	14.2147	DCON12	0.255	DCON12	0.255	No Messages	No Messages
76	3' PILE	Column	Design	No Messages	60	DCON11	13.579	DCON12	0.255	DCON12	0.255	No Messages	No Messages
77	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
77	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
77	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
78	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
78	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
78	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
79	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
79	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
79	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
80	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
80	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
80	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
81	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
81	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
81	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
82	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
82	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
82	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
93	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
93	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
93	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
94	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
94	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
94	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
95	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
95	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
95	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages

96	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
96	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
96	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
97	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
97	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
97	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
98	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
98	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
98	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
99	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
99	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
99	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
100	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
100	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
100	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
101	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
101	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
101	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
102	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
102	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
102	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
103	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
103	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
103	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
104	3' PILE	Column	Design	No Messages	0	DCON12	13.5981	DCON12	0.255	DCON12	0.255	No Messages	No Messages
104	3' PILE	Column	Design	No Messages	15	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
104	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
105	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
105	3' PILE	Column	Design	No Messages	15	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
105	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
106	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
106	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
106	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
107	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
107	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
107	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
108	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
108	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
108	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
109	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
109	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
109	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
110	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
110	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
110	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
111	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
111	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
111	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
112	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
112	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
112	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
113	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
113	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
113	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
114	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
114	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
114	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
115	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
115	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
115	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
116	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
116	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
116	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
117	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
117	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
117	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
118	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
118	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
118	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
119	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
119	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
119	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
120	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
120	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
120	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
121	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
121	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
121	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
122	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
122	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
122	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
123	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
123	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
123	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages

124	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
124	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
124	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
125	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
125	3' PILE	Column	Design	No Messages	15	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
125	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
126	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
126	3' PILE	Column	Design	No Messages	15	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
126	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
127	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
127	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
127	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
128	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
128	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
128	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
129	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
129	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
129	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
130	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
130	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
130	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
131	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
131	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
131	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
132	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
132	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
132	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
133	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
133	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
133	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
134	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
134	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
134	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
135	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
135	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
135	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
136	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
136	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
136	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
137	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
137	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
137	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
138	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
138	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
138	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
139	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
139	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
139	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
140	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
140	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
140	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
141	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
141	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
141	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
142	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
142	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
142	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
143	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
143	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
143	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
144	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
144	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
144	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
145	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
145	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
145	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
146	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
146	3' PILE	Column	Design	No Messages	15	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
146	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
147	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
147	3' PILE	Column	Design	No Messages	15	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
147	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
148	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
148	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
148	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
149	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
149	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
149	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
150	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
150	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
150	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
151	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
151	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
151	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages

152	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
152	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
152	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
153	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
153	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
153	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
154	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
154	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
154	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
155	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
155	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
155	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
156	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
156	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
156	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
157	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
157	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
157	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
158	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
158	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
158	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
159	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
159	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
159	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
160	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
160	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
160	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
161	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
161	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
161	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
162	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
162	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
162	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
163	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
163	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
163	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
164	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
164	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
164	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
165	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
165	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
165	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
166	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
166	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
166	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
167	3' PILE	Column	Design	No Messages	0	DCON12	13.5991	DCON12	0.255	DCON12	0.255	No Messages	No Messages
167	3' PILE	Column	Design	No Messages	15	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
167	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
168	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
168	3' PILE	Column	Design	No Messages	15	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
168	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
169	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
169	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
169	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
170	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
170	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
170	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
171	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
171	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
171	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
172	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
172	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
172	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
173	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
173	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
173	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
174	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
174	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
174	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
175	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
175	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
175	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
176	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
176	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
176	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
177	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
177	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
177	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
178	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
178	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
178	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
179	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
179	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
179	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages

180	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
180	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
180	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
181	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
181	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
181	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
182	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
182	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
182	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
183	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
183	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
183	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
184	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
184	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
184	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
185	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
185	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
185	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
186	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
186	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
186	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
187	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
187	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
187	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
5	3' PILE	Column	Design	No Messages	0	DCON11	17.2311	DCON12	0.255	DCON12	0.255	No Messages	No Messages
5	3' PILE	Column	Design	No Messages	15	DCON11	14.0863	DCON12	0.255	DCON12	0.255	No Messages	No Messages
5	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
6	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
6	3' PILE	Column	Design	No Messages	15	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
6	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
7	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
7	3' PILE	Column	Design	No Messages	30	DCON11	14.1131	DCON12	0.255	DCON12	0.255	No Messages	No Messages
7	3' PILE	Column	Design	No Messages	60	DCON11	16.7266	DCON12	0.255	DCON12	0.255	No Messages	No Messages
9	3' PILE	Column	Design	No Messages	0	DCON11	14.8154	DCON12	0.255	DCON12	0.255	No Messages	No Messages
9	3' PILE	Column	Design	No Messages	30	DCON11	14.2143	DCON12	0.255	DCON12	0.255	No Messages	No Messages
9	3' PILE	Column	Design	No Messages	60	DCON11	13.5775	DCON12	0.255	DCON12	0.255	No Messages	No Messages
11	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
11	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
11	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
12	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
12	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
12	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
13	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
13	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
13	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
33	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
33	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
33	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
34	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
34	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
34	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
35	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
35	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
35	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
36	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
36	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
36	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
37	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
37	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
37	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
38	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
38	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
38	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
39	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
39	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
39	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
40	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
40	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
40	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
41	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
41	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
41	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
42	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
42	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
42	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
43	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
43	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
43	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
44	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
44	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
44	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
45	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
45	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
45	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages

46	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
46	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
46	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
47	3' PILE	Column	Design	No Messages	0	DCON11	17.1782	DCON12	0.255	DCON12	0.255	No Messages	No Messages
47	3' PILE	Column	Design	No Messages	15	DCON11	14.0112	DCON12	0.255	DCON12	0.255	No Messages	No Messages
47	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
48	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
48	3' PILE	Column	Design	No Messages	15	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
48	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
49	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
49	3' PILE	Column	Design	No Messages	30	DCON11	14.0375	DCON12	0.255	DCON12	0.255	No Messages	No Messages
49	3' PILE	Column	Design	No Messages	60	DCON11	16.6702	DCON12	0.255	DCON12	0.255	No Messages	No Messages
50	3' PILE	Column	Design	No Messages	0	DCON11	14.7559	DCON12	0.255	DCON12	0.255	No Messages	No Messages
50	3' PILE	Column	Design	No Messages	30	DCON11	14.1518	DCON12	0.255	DCON12	0.255	No Messages	No Messages
50	3' PILE	Column	Design	No Messages	60	DCON11	13.5117	DCON12	0.255	DCON12	0.255	No Messages	No Messages
51	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
51	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
51	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
188	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
188	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
188	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
189	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
189	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
189	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
190	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
190	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
190	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
191	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
191	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
191	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
192	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
192	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
192	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
193	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
193	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
193	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
194	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
194	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
194	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
195	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
195	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
195	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
196	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
196	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
196	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
197	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
197	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
197	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
198	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
198	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
198	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
199	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
199	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
199	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
200	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
200	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
200	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
201	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
201	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
201	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
202	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
202	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
202	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
203	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
203	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
203	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
204	3' PILE	Column	Design	No Messages	0	DCON11	17.2399	DCON12	0.255	DCON12	0.255	No Messages	No Messages
204	3' PILE	Column	Design	No Messages	15	DCON11	14.0937	DCON12	0.255	DCON12	0.255	No Messages	No Messages
204	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
205	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
205	3' PILE	Column	Design	No Messages	15	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
205	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
206	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
206	3' PILE	Column	Design	No Messages	30	DCON11	14.1074	DCON12	0.255	DCON12	0.255	No Messages	No Messages
206	3' PILE	Column	Design	No Messages	60	DCON11	16.7259	DCON12	0.255	DCON12	0.255	No Messages	No Messages
207	3' PILE	Column	Design	No Messages	0	DCON11	14.8148	DCON12	0.255	DCON12	0.255	No Messages	No Messages
207	3' PILE	Column	Design	No Messages	30	DCON11	14.2147	DCON12	0.255	DCON12	0.255	No Messages	No Messages
207	3' PILE	Column	Design	No Messages	60	DCON11	13.579	DCON12	0.255	DCON12	0.255	No Messages	No Messages
208	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
208	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
208	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
209	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
209	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
209	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages

210	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
210	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
210	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
211	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
211	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
211	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
212	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
212	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
212	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
213	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
213	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
213	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
214	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
214	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
214	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
215	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
215	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
215	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
216	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
216	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
216	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
217	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
217	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
217	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
218	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
218	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
218	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
219	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
219	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
219	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
220	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
220	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
220	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
221	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
221	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
221	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
222	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
222	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
222	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
223	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
223	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
223	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
224	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
224	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
224	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
225	3' PILE	Column	Design	No Messages	0	DCON12	13.5981	DCON12	0.255	DCON12	0.255	No Messages	No Messages
225	3' PILE	Column	Design	No Messages	15	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
225	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
226	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
226	3' PILE	Column	Design	No Messages	15	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
226	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
227	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
227	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
227	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
228	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
228	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
228	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
229	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
229	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
229	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
230	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
230	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
230	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
231	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
231	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
231	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
232	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
232	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
232	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
233	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
233	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
233	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
234	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
234	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
234	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
235	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
235	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
235	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
236	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
236	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
236	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
237	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
237	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
237	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages

238	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
238	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
238	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
239	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
239	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
239	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
240	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
240	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
240	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
241	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
241	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
241	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
242	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
242	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
242	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
243	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
243	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
243	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
244	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
244	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
244	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
245	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
245	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
245	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
246	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
246	3' PILE	Column	Design	No Messages	15	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
246	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
247	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
247	3' PILE	Column	Design	No Messages	15	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
247	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
248	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
248	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
248	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
249	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
249	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
249	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
250	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
250	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
250	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
251	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
251	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
251	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
252	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
252	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
252	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
253	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
253	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
253	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
254	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
254	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
254	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
255	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
255	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
255	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
256	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
256	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
256	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
257	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
257	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
257	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
258	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
258	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
258	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
259	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
259	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
259	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
260	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
260	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
260	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
261	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
261	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
261	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
262	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
262	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
262	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
263	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
263	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
263	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
264	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
264	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
264	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
265	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
265	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
265	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages

266	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
266	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
266	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
267	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
267	3' PILE	Column	Design	No Messages	15	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
267	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
268	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
268	3' PILE	Column	Design	No Messages	15	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
268	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
269	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
269	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
269	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
270	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
270	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
270	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
271	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
271	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
271	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
272	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
272	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
272	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
273	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
273	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
273	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
274	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
274	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
274	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
275	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
275	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
275	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
276	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
276	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
276	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
277	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
277	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
277	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
278	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
278	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
278	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
279	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
279	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
279	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
280	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
280	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
280	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
281	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
281	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
281	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
282	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
282	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
282	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
283	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
283	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
283	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
284	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
284	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
284	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
285	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
285	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
285	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
286	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
286	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
286	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
287	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
287	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
287	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
288	3' PILE	Column	Design	No Messages	0	DCON12	13.5991	DCON12	0.255	DCON12	0.255	No Messages	No Messages
288	3' PILE	Column	Design	No Messages	15	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
288	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
289	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
289	3' PILE	Column	Design	No Messages	15	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
289	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
290	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
290	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
290	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
291	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
291	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
291	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
292	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
292	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
292	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
293	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
293	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
293	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages

294	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
294	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
294	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
295	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
295	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
295	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
296	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
296	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
296	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
297	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
297	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
297	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
298	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
298	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
298	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
299	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
299	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
299	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
300	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
300	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
300	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
301	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
301	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
301	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
302	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
302	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
302	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
303	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
303	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
303	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
304	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
304	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
304	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
305	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
305	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
305	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
306	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
306	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
306	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
307	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
307	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
307	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
308	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
308	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
308	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
314	3x3 COLUMN	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
314	3x3 COLUMN	Column	Design	No Messages	12	DCON12	12.96	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
314	3x3 COLUMN	Column	Design	No Messages	24	DCON12	12.96	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
315	3x3 COLUMN	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
315	3x3 COLUMN	Column	Design	No Messages	12	DCON12	12.96	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
315	3x3 COLUMN	Column	Design	No Messages	24	DCON12	14.9472	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
316	3x3 COLUMN	Column	Design	No Messages	0	DCON12	14.9515	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
316	3x3 COLUMN	Column	Design	No Messages	12	DCON12	18.8791	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
316	3x3 COLUMN	Column	Design	No Messages	24	DCON11	21.6161	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
317	3x3 COLUMN	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
317	3x3 COLUMN	Column	Design	No Messages	12	DCON12	12.96	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
317	3x3 COLUMN	Column	Design	No Messages	24	DCON12	12.96	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
318	3x3 COLUMN	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
318	3x3 COLUMN	Column	Design	No Messages	12	DCON12	12.96	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
318	3x3 COLUMN	Column	Design	No Messages	24	DCON11	14.1641	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
319	3x3 COLUMN	Column	Design	No Messages	0	DCON11	14.1651	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
319	3x3 COLUMN	Column	Design	No Messages	12	DCON11	18.5354	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
319	3x3 COLUMN	Column	Design	No Messages	24	DCON11	21.4601	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
320	3x3 COLUMN	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
320	3x3 COLUMN	Column	Design	No Messages	12	DCON12	12.96	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
320	3x3 COLUMN	Column	Design	No Messages	24	DCON12	12.96	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
321	3x3 COLUMN	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
321	3x3 COLUMN	Column	Design	No Messages	12	DCON12	12.96	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
321	3x3 COLUMN	Column	Design	No Messages	24	DCON11	14.1591	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
322	3x3 COLUMN	Column	Design	No Messages	0	DCON11	14.1602	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
322	3x3 COLUMN	Column	Design	No Messages	12	DCON11	18.5314	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
322	3x3 COLUMN	Column	Design	No Messages	24	DCON11	21.4568	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
323	3x3 COLUMN	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
323	3x3 COLUMN	Column	Design	No Messages	12	DCON12	12.96	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
323	3x3 COLUMN	Column	Design	No Messages	24	DCON12	12.96	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
324	3x3 COLUMN	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
324	3x3 COLUMN	Column	Design	No Messages	12	DCON12	12.96	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
324	3x3 COLUMN	Column	Design	No Messages	24	DCON12	14.9477	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
325	3x3 COLUMN	Column	Design	No Messages	0	DCON12	14.952	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
325	3x3 COLUMN	Column	Design	No Messages	12	DCON12	18.8794	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
325	3x3 COLUMN	Column	Design	No Messages	24	DCON11	21.6063	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
326	3x3 COLUMN	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
326	3x3 COLUMN	Column	Design	No Messages	12	DCON12	12.96	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
326	3x3 COLUMN	Column	Design	No Messages	24	DCON12	12.96	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages

327	3x3 COLUMN	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
327	3x3 COLUMN	Column	Design	No Messages	12	DCON12	12.96	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
327	3x3 COLUMN	Column	Design	No Messages	24	DCON12	14.9472	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
328	3x3 COLUMN	Column	Design	No Messages	0	DCON12	14.9515	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
328	3x3 COLUMN	Column	Design	No Messages	12	DCON12	18.8791	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
328	3x3 COLUMN	Column	Design	No Messages	24	DCON11	21.6161	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
329	3x3 COLUMN	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
329	3x3 COLUMN	Column	Design	No Messages	12	DCON12	12.96	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
329	3x3 COLUMN	Column	Design	No Messages	24	DCON12	12.96	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
330	3x3 COLUMN	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
330	3x3 COLUMN	Column	Design	No Messages	12	DCON12	12.96	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
330	3x3 COLUMN	Column	Design	No Messages	24	DCON11	14.1641	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
331	3x3 COLUMN	Column	Design	No Messages	0	DCON11	14.1651	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
331	3x3 COLUMN	Column	Design	No Messages	12	DCON11	18.5354	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
331	3x3 COLUMN	Column	Design	No Messages	24	DCON11	21.4601	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
332	3x3 COLUMN	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
332	3x3 COLUMN	Column	Design	No Messages	12	DCON12	12.96	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
332	3x3 COLUMN	Column	Design	No Messages	24	DCON12	12.96	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
333	3x3 COLUMN	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
333	3x3 COLUMN	Column	Design	No Messages	12	DCON12	12.96	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
333	3x3 COLUMN	Column	Design	No Messages	24	DCON11	14.1591	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
334	3x3 COLUMN	Column	Design	No Messages	0	DCON11	14.1602	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
334	3x3 COLUMN	Column	Design	No Messages	12	DCON11	18.5314	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
334	3x3 COLUMN	Column	Design	No Messages	24	DCON11	21.4568	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
335	3x3 COLUMN	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
335	3x3 COLUMN	Column	Design	No Messages	12	DCON12	12.96	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
335	3x3 COLUMN	Column	Design	No Messages	24	DCON12	12.96	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
336	3x3 COLUMN	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
336	3x3 COLUMN	Column	Design	No Messages	12	DCON12	12.96	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
336	3x3 COLUMN	Column	Design	No Messages	24	DCON12	14.9477	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
337	3x3 COLUMN	Column	Design	No Messages	0	DCON12	14.952	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
337	3x3 COLUMN	Column	Design	No Messages	12	DCON12	18.8794	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
337	3x3 COLUMN	Column	Design	No Messages	24	DCON11	21.6063	DCON12	0.3825	DCON12	0.3825	No Messages	No Messages
1	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
1	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
1	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
2	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
2	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
2	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
3	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
3	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
3	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
4	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
4	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
4	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
83	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
83	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
83	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
309	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
309	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
309	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
310	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
310	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
310	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
441	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
441	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
441	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
442	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
442	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
442	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
443	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
443	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
443	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
444	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
444	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
444	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
445	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
445	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
445	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
446	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
446	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
446	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
447	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
447	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
447	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
448	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
448	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
448	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
449	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
449	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
449	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
450	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
450	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
450	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages

451	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
451	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
451	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
452	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
452	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
452	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
453	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
453	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
453	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
454	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
454	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
454	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
455	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
455	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
455	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
456	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
456	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
456	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
457	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
457	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
457	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
458	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
458	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
458	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
459	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
459	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
459	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
460	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
460	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
460	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
461	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
461	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
461	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
462	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
462	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
462	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
463	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
463	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
463	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
464	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
464	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
464	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
465	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
465	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
465	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
466	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
466	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
466	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
467	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
467	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
467	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
468	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
468	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
468	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
469	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
469	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
469	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
470	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
470	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
470	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
471	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
471	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
471	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
472	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
472	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
472	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
473	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
473	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
473	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
474	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
474	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
474	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
475	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
475	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
475	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
476	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
476	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
476	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
477	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
477	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
477	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
478	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
478	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
478	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages

479	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
479	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
479	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
480	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
480	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
480	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
481	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
481	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
481	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
482	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
482	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
482	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
483	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
483	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
483	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
484	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
484	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
484	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
485	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
485	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
485	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
486	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
486	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
486	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
487	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
487	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
487	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
488	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
488	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
488	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
489	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
489	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
489	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
490	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
490	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
490	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
491	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
491	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
491	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
492	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
492	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
492	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
493	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
493	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
493	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
494	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
494	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
494	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
495	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
495	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
495	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
496	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
496	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
496	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
497	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
497	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
497	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
498	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
498	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
498	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
499	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
499	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
499	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
500	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
500	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
500	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
501	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
501	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
501	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
502	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
502	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
502	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
503	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
503	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
503	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
504	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
504	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
504	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
505	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
505	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
505	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
506	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
506	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
506	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages

507	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
507	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
507	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
508	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
508	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
508	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
509	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
509	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
509	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
510	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
510	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
510	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
511	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
511	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
511	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
512	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
512	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
512	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
513	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
513	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
513	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
514	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
514	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
514	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
515	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
515	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
515	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
516	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
516	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
516	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
517	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
517	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
517	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
518	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
518	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
518	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
519	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
519	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
519	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
520	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
520	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
520	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
521	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
521	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
521	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
522	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
522	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
522	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
523	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
523	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
523	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
524	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
524	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
524	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
525	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
525	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
525	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
526	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
526	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
526	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
527	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
527	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
527	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
528	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
528	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
528	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
529	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
529	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
529	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
530	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
530	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
530	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
531	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
531	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
531	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
532	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
532	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
532	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
533	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
533	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
533	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
534	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
534	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
534	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages

535	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
535	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
535	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
536	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
536	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
536	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
537	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
537	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
537	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
538	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
538	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
538	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
539	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
539	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
539	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
540	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
540	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
540	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
541	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
541	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
541	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
542	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
542	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
542	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
543	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
543	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
543	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
544	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
544	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
544	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
545	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
545	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
545	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
546	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
546	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
546	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
547	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
547	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
547	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
548	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
548	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
548	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
549	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
549	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
549	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
550	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
550	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
550	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
551	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
551	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
551	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
552	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
552	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
552	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
553	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
553	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
553	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
554	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
554	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
554	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
555	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
555	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
555	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
556	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
556	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
556	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
557	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
557	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
557	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
558	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
558	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
558	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
559	3' PILE	Column	Design	No Messages	0	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
559	3' PILE	Column	Design	No Messages	30	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages
559	3' PILE	Column	Design	No Messages	60	DCON12	12.96	DCON12	0.255	DCON12	0.255	No Messages	No Messages